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THE BASIC PRINCIPLES USED IN THE DESIGNS FOR THE NEW WATER SUPPLY WORKS OF WINNIPEG, MANITOBA[1]

By James H. Fuertes[2]

Winnipeg is situated about 65 miles north of the international boundary, and about 35 miles south of the southern end of Lake Winnipeg, at the junction of the Red River, which rises in Minnesota and follows a northerly course, and the Assiniboine River joining it from the west, the combined waters continuing north to an outlet into Lake Winnipeg. The city, prior to its organization as a town, was merely a small settlement or trading post, called Fort Garry, at the junction of these two rivers. In 1874 the population of Winnipeg is given as 1869; by 1890 it had jumped to 23,000; by 1910 to 132,720. By 1913 the population of the entire Greater Winnipeg Water District was 215,000; the present population is estimated to be 250,000.

History. The new water works of Winnipeg, forming the subject of this paper, were built to provide a satisfactorily large supply of soft water. Before the construction of these new works the city depended upon a ground water supply of very hard water, unfit, without softening, for manufacturing or commercial uses, and too limited in quantity even for domestic use. Under such conditions the establishment at Winnipeg of manufacturing or industrial works requiring a plentiful supply of suitable water was out of the question and a great handicap to the proper development of her otherwise excellent resources as a trade center. The question of changing the water supply had been agitated annually for a number of years, but the inevitable expense involved in going a great distance to secure a new supply necessitated the postponement of active steps in that direction a number of times.

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[1] Read at the Montreal Convention, June 23, 1920. A supplementary paper read at the same time by Wm. G. Chase, describing the construction of the Winnipeg System, will be printed later. Discussion of this paper is requested and should be sent to the Editor.

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Prior to 1880 the municipal water supply of Winnipeg was obtained from wells scattered about the town, and was distributed to the consumers from tanks and barrels on carts in the summer, or sleds in the winter. In 1882 the Winnipeg Water Works Company, operating under an exclusive twenty year franchise, built a water works plant on the Assiniboine River on Armstrong Point, establishing a pumping station and mechanical filter plant, of the pressure type, the filtered water being pumped directly into the city's street mains. This system was purchased by the city in April, 1899, for a little over a quarter of a million dollars, but was abandoned in 1905 following the introduction of a ground water supply.

The quality of the water purveyed from the Assiniboine plant was not satisfactory, nor was the ground water supply, both being very hard. Agitation in favor of a new and better supply was kept up more or less continuously from that time until the commencement of the building of the present new supply.

In March, 1883, Dr. Agnew in a letter to the *Free Press* directed attention to the Lake of the Woods as a source to which Winnipeg must ultimately look for her water supply, and again elaborated upon this source in February, 1884, in an address before the Manitoba Historical and Scientific Society. In February, 1895, Walter Moberley, C.E., made a report to the City Council advocating a new supply from the Winnipeg River. Nothing was done with either suggestion, however, and in October, 1896, Col. H. N. Ruttan, city engineer, reported on the relative merits of a supply from the Assiniboine River and a supply from artesian wells. The well supply was developed quite extensively; the water, however, while agreeable to the taste and excellent as to appearance, was too hard for ordinary municipal purposes and in September, 1897, Dr. Rudolph Hering was asked to report upon the possibility of softening this water and on the merits of other available sources of supply. Dr. Hering examined and reported on a ground water supply from Poplar Springs, a pumped supply from the Assiniboine River, a pumped supply from Winnipeg River, and on the extension and softening of the artesian well supply, recommending that the softened ground water supply be adopted and further developed.

About 1905, the city, having grown very rapidly in population and the ground water supply having been much depleted, so far as local sources were concerned, another examination was made and reported on to Councils in 1907 by Messrs. Fuertes, Lea, Schwitzer and Whipple, who investigated the possibilities of extending the ground water supply, securing a supply from the Red River and the Assiniboine River, as well as supplies from the Winnipeg River, and from the Lake of the Woods. This report favored developing a supply from Winnipeg River as being less expensive than a supply from the Lake of the Woods, while being equally satisfactory, after proper treatment. Following this report another investigation was made by Prof. C. S. Slichter for Public Utility Commissioner H. A. Robson, favoring a supply from the Lake of the Woods or Shoal Lake; and in the following year, 1913, the question of the desirability and availability of the Shoal Lake supply was again referred to a commission of engineers composed of Dr. Rudolph Hering, Frederick P. Stearns and James H. Fuertes, who after considering the possibilities of a gravity supply to Winnipeg from the Lake of the Woods, through a concrete aqueduct, concluded that works could be built of much larger capacity than had heretofore been considered for a sum which it would not be too difficult for the district to provide. The report was adopted by the officials of the Greater Winnipeg Water District on September 6, 1913, by the City Council of Winnipeg on September 8, 1913, and voted on affirmatively by popular vote on October 1, 1913. On October 20 five field parties were on the ground making the preliminary surveys for the alignment.

THE NEW SUPPLY

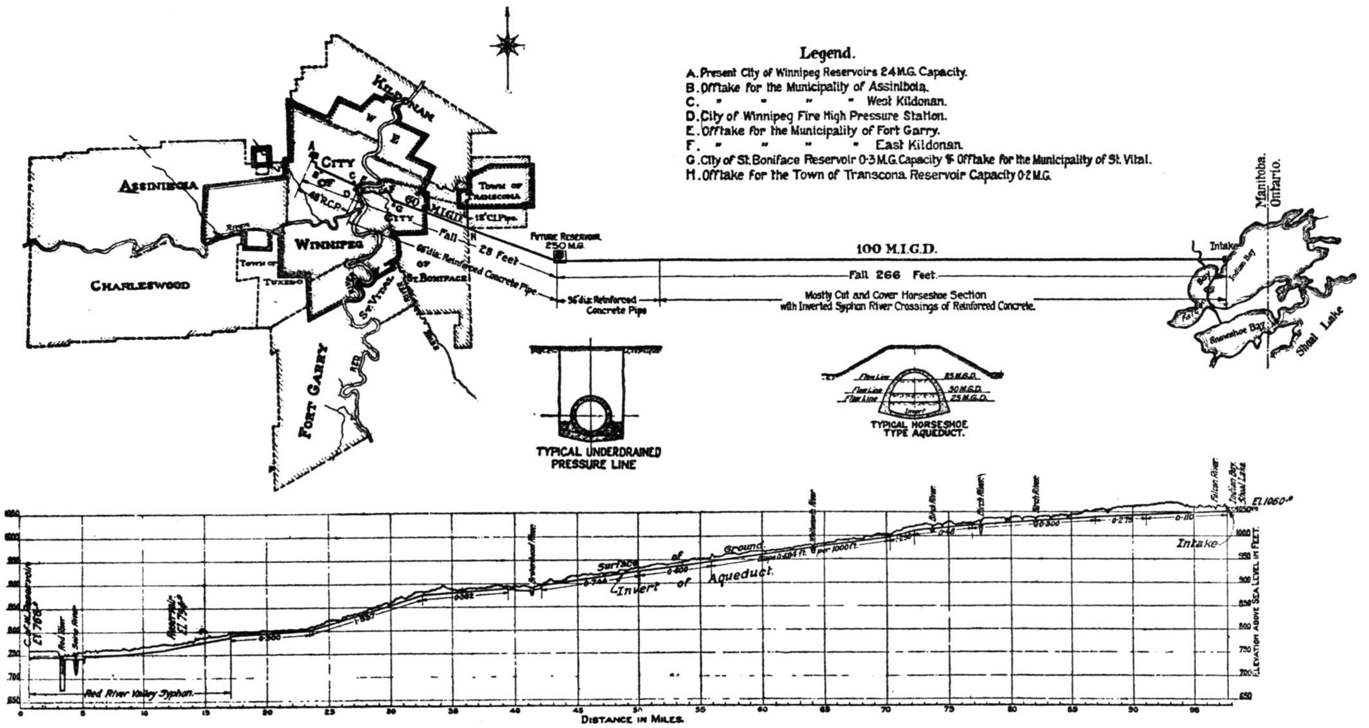
Organization. Since Shoal Lake waters are tributary to the waters of the Lake of the Woods, a portion of which crosses the international boundary into the United States, it was necessary to secure for the project the approval of the International Joint Commission having jurisdiction over boundary waters; and as the boundary line between the provinces of Manitoba and Ontario passed through Indian Bay, a tributary of Shoal Lake, it was necessary also to secure the consent of the Ontario Government to the taking of these waters.

The Greater Winnipeg Water District, consisting of the city of Winnipeg and six smaller neighboring municipalities, was organized and constituted by proclamation of the lieutenant governor of Manitoba, June 10, 1913. The Water District was authorized to go outside of Manitoba for water by the Dominion Parliament in Act 3–4, George V, Chapter 208, and on October 2, 1913, an order in Council of the Province of Ontario was passed permitting the use of water for the Winnipeg Water District from Shoal Lake up to a limit of 100,000,000 gallons per day. The International Joint Commission approved the application January 15, 1914.

All difficulties in regard to the securing of the water having been settled, surveys and investigations were immediately started and by the end of February, 1914, the location was completed, there having been involved the taking of 380 square miles of topography, 362 miles of transit lines, 1,317 miles of levels, 95 miles of precise levels, 11,544 feet of soundings in Indian Bay, and 3,897 feet of test borings along the line of the aqueduct. The work of clearing the right-of-way was started in March, 1914, and finished in about three months, the standard width of the right-of-way being 300 feet; greater widths were secured where necessary. The first actual construction work was the erection of the telephone line, 91 miles long, which was begun May 5, 1914, and cost about $32,500. In October, 1914, contracts were awarded for the whole aqueduct from Deacon to Shoal Lake. The first water was turned through the completed aqueduct and discharged into the McPhillips street reservoir in Winnipeg on March 26, 1919. The general features of the system are outlined in figure 1.

General description of country traversed by the aqueduct. It was found, from studies and inspection, that the line proposed in the Hering, Stearns, Fuertes report, which was based partly upon surveys made specially for that report, and partly upon interpolated profiles based upon the profiles of the Canadian Pacific and Grand Trunk Railways, was a practicable line, but that a more intimate knowledge of the country on either side of this line gave promise of securing a more economical aqueduct than that following the line proposed. Throughout the whole length of the line the country is very flat and was largely covered with swamps, timber and underbrush. Only in a few places was it possible to see off to any distance from the line, and actual elevations had to be taken along the section lines, or wherever clearings had been cut out, for a distance, sometimes, as much as ten miles either side of the line, in order to avoid running into impracticable country on the one hand, and in order to be able to pick out a better alignment on the other hand.

For about 20 miles eastward from Winnipeg, the plains, formed by the deposition of clays from suspension in the sea water of the ancient glacial Lake Agassiz, have a slope of but 1 to 3 feet to the mile. To the eastward of this plain, the country rises somewhat more rapidly for a few miles. Large deposits of gravel occur in


Legend.
A. Present City of Winnipeg Reservoirs 24 M.G. Capacity.
B. Offtake for the Municipality of Assiniboia.
C. " " " " West Kildonan.
D. City of Winnipeg Fire High Pressure Station.
E. Offtake for the Municipality of Fort Garry.
F. " " " " East Kildonan.
G. City of St. Boniface Reservoir 0·3 M.G. Capacity & Offtake for the Municipality of St. Vital.
H. Offtake for the Town of Transcona Reservoir Capacity 0·2 M.G.
Assiniboia
Charleswood
Kildonan
W
E
City of Winnipeg
A
B
C
D
F
G
H
48" R.C.P.
60 M.I.G.D.
Town of Transcona
12" C.I. Pipe.
City of St. Boniface
Town of Tuxedo
River
St. Vital
Fort Garry
Red River
Seine River
Fall 28 Feet
60" dia. Reinforced Concrete Pipe
Future Reservoir 250 M.G.
36" dia. Reinforced Concrete Pipe
100 M.I.G.D.
Fall 266 Feet.
Mostly Cut and Cover Horseshoe Section with Inverted Syphon River Crossings of Reinforced Concrete.
Intake
Indian Bay
Bay
Falcon
Snowshoe Bay
Shoal Lake
Manitoba.
Ontario.
TYPICAL UNDERDRAINED PRESSURE LINE
Flow Line 85 M.G.D.
Flow Line 50 M.G.D.
Flow Line 25 M.G.D.
Invert
TYPICAL HORSESHOE TYPE AQUEDUCT.
C. of W. Reservoir El. 766
Red River
Seine River
Reservoir El. 794
Red River Valley Syphon.
0·300
1·557
0·382
Brokenhead River
0·744
0·600
Surface of Ground
Invert of Aqueduct.
Slope 0·694 ft. per 1000 ft.
Whitemouth River
1·250
Birch River
0·48
Birch River
Birch River
0·300
0·275
0·110
Falcon River
Indian Bay Shoal Lake
El. 1060
Intake
1050
1000
950
900
850
800
750
700
650
Elevation above Sea Level in Feet.
0
5
10
15
20
25
30
35
40
45
50
55
60
65
70
75
80
85
90
95
Distance in Miles.

Total Fall All Available for Gravity Delivery 294 Feet.

FIG. 1. GENERAL FEATURES OF THE GREATER WINNIPEG WATER SUPPLY WORKS

ridges along what were, in ancient times, progressive shore lines of this inland sea as the continental ice-cap receded. Where the shore line followed along the contour of the ground, the glacial deposits were washed out leaving the sand and gravel comparatively clean and free from clay. Going still further east and, in fact, located more or less along the whole length of the aqueduct, the soil is a mixture of clay, sand, and gravel, sometimes stratified, sometimes mixed, on top of which has grown up successive generations of vegetation forming what are locally termed "muskegs." The sub-soil throughout the whole distance is practically water-tight, and consequently the run-off of rain and melting snow, which amount together to only about 22 inches from the watershed in a year, is retarded so greatly that but three open streams are crossed by the aqueduct in the nearly 100 miles between Shoal Lake and the Seine River, which joins Red River at Winnipeg.

Source of supply. The report of Hering, Stearns and Fuertes recommended securing the water from Indian Bay, that portion of the Lake of the Woods nearest to Winnipeg. Indian Bay is an arm of Shoal Lake, and Shoal Lake, which has a water area of 110 square miles, is connected with the main Lake of the Woods by means of a narrow channel called Ash Rapids. While the watershed of Shoal Lake, 360 square miles, is not large enough to maintain in very dry years a yield of 85,000,000 gallons a day, this yield can be had by drawing through it on the main body of water in the Lake of the Woods without appreciably affecting the water level of the main lake. The fluctuation in level between low and high water in Indian Bay or Shoal Lake lies between elevations 1059.6 and 1065.0 above sea level.

Indian Bay, from which the water is taken, is about 6 miles long and from 1 to 3 miles wide. Numbers of soundings lengthwise and crosswise of this lake show its depth to vary from 24 to 26 feet over the whole bottom, except in the immediate vicinity of the shores, which shoal up more or less rapidly. At the western end of Indian Bay a stream known as Falcon River enters and discharges the highly colored swamp waters lying to the northwest of Indian Bay. Just to the south of and parallel with Indian Bay lies Snowshoe Bay. A promontory varying from one-half a mile to a mile or more in width, and with a length of about six miles, separates Snowshoe Bay from Indian Bay. Both Snowshoe and Indian Bays connect openly with Shoal Lake at the eastern end of the promontory above mentioned.

Decolorization. The waters of all the bays, tributary to Lake of the Woods, are more or less highly colored, being fed by small streams having their origin in the swamps and muskegs bordering the main lake. The color of the waters of Shoal Lake and of the main Lake of the Woods, away from the influence of these small highly colored streams, and where time and the bleaching action of sunlight have had an opportunity to bring about decolorization, is not high enough to be objectionable for ordinary municipal uses. Indian Bay, when first examined, was decidedly discolored throughout its whole area, the greater part of the discoloration coming from Falcon River.

A study of the topography of the promontory lying between Snowshoe and Indian Bays disclosed a low area through which a canal could be cut at small expense, to connect the two lakes, which suggested the plan, subsequently adopted, of diverting the black waters of Snake Lake and Falcon River through this canal into Snowshoe Bay by the building of a dyke across the shallow water at the west end of Indian Bay. The cost of this dyke and canal, which secured an intake point from which low colored water could be had in Indian Bay, was about $147,000; to extend this aqueduct 5 miles further, to Shoal Lake, the only other alternative to secure satisfactory water, would have cost about $1,000,000. The results have been quite satisfactory, as may be seen from the statement that on June 7, 1915, the diversion dyke having been completed in 1914, the color of the water on the Falcon River side of the dyke across the end of Indian Bay was 107 while the color on the Indian Bay side of the dyke on the same day was 9 (platinum cobalt scale).

The natural color of the water in Indian Bay is lower in the winter than in the summer; under the ice, the color remains practically constant during the entire winter. During the winter of 1919–1920 the color at the intake remained at 12.

The bottom of Indian Bay is more or less covered with accumulations of leaves, shells of infusoria, dead leaves, grasses and other matters of organic and mineral origin, generally reduced by time and natural agencies to a stable or non-putrescible condition. In addition, the waters contain numbers of algae and living organisms of various kinds. The lake being comparatively shallow, the waters experience two distinct turnovers each year, due to the changes in temperature above and below the temperature of maximum density. Ice forms on the lake to a thickness of about 4 feet and the temperature of the water drops to about 34° at the bottom of the lake in very long continued cold weather. When water, at the surface of the lake, on the approach of cold weather reaches a temperature of 39° it tends to sink to the bottom of the lake and displace the warmer and lighter water underneath. This continues until the water of the entire lake has reached a graded temperature such that the heaviest remains at the bottom and the lightest at the top. In the following Spring, when the surface water again warms up, the warmth being transmitted gradually to the lower strata, as soon as that at the surface reaches a temperature of 39° it sinks, as before, displacing the colder water beneath, and a gradual readjustment of the entire mass takes place.

There are thus two distinct periods when bottom water will be brought to the surface, and with each over-turning, of course, some of the matters which have settled to the bottom will be caught in the rising currents and brought up to the surface. This phenomenon is one which required watching for many reasons, one of which was discovered during the first year of operation of the new works. On the occasion referred to, such large quantities of grasses and weeds were brought up during the over-turn that the intake screens, before thought was given the matter by the local attendant, became clogged sufficiently to cause a head of several inches on the screens, with the consequent breaking out of the wire mesh, permitting dirty water and fish to pass into the aqueduct.

BASIS OF DESIGN FOR WORKS

Design of intake. The design of the lake intake was studied out with a view to preventing troubles that might interrupt the supply of water to the city. These troubles, it was anticipated, would be due largely to ice in the extremely cold winter weather. It was planned, therefore, to allow for an ice sheet 4 feet thick on the lake and to have a passage beneath this sufficiently wide and deep enough to admit the water into the intake with as little loss of head as possible and without drawing in spicules of ice forming on the under side of the ice sheet.

The general plan was to extend two rock ballasted earthen dykes, with side slopes of 1 on 3, in parallel lines out from the shore 200 feet to where the water was about 22 feet deep, at high water, the dykes being 180 feet apart on their center lines The space between these dykes was dredged out and the whole area covered with screened gravel, bringing the finished surface up to elevation 1048, or about 13 feet below mean lake level. The front wall of the concrete gate house was carried down to a depth of 8.7 feet below average lake level, or 6.25 feet below low water, in order to cut off the entrance of cold air into the screen chamber through the intake, and in order to prevent the formation of ice in the intake structure. The screen chamber itself was simply an extension of the aqueduct with its bottom widened and its sides made vertical. This chamber was divided into two parts by a longitudinal partition and two sets of screens were placed within in such a way as to offer a large area to the moving water and permit of removal for cleaning. The whole screen house was covered with an earth embankment 4 feet deep for frost-proofing, so that ice would not form on the fine mesh screens and retard the passage of the water.

Air temperatures as low as 50° below zero are not infrequent at the intake and commonly temperatures of zero and lower prevail continuously from the beginning of December until the end of March. Frost, therefore, was a matter to be reckoned with in the design for these works. The general arrangement of the intake structure is shown in figure 2.

The screens had a total submerged area at high water of about 700 square feet, which, for a water consumption of 100,000,000 gallons[3] per day would correspond with 7 square feet per million gallons per day. The screens were of copper wire cloth having ⅜-inch mesh backed by a screen with 1-inch meshes. After the accident of the breaking through of the screens last summer, an additional set of vertical steel racks with 1½ inch spacing was installed just below the upper stop log slot; a second set of screens with six meshes to the inch was also installed in connection with the coarser screen to guard against a possible repetition of that unfortunate and unpleasant experience. The three barriers have been successful in excluding undesirable water-borne life.

The screens are lifted for cleaning by a chain hoist running on overhead tracks. Inside the screen house below the screens, there is a boat entrance to permit the dropping of a boat into the aqueduct for the purposes of inspection from the lake to the inverted siphon at the Venturi meter under Falcon River. Another boat entrance is placed just beyond Falcon River to permit of inspection from Falcon River to Birch River, and again, at the beginning and end of each of the inverted siphons to the end of the arched section 5 miles west of Deacon.

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[3] The imperial gallon, not the United States gallon, is the unit employed in Canada and referred to in this paper.

Temperature of water flowing through aqueduct. As to temperatures of the water at Indian Bay, as compared with the temperatures in the city reservoir after passage through the 100 miles of aqueduct, it may be remarked that there is practically no difference at any season of the year. The temperature at the intake, as well as at the city reservoir, gradually decreases, from the time the ice begins to form on the lake in November until a minimum of 2°C. (35.5°F.) is reached in December at the reservoir, persisting until March; from the time of the break-up in the spring, generally in April, there is a gradual rise in temperature until a maximum of 22.5°C. (72.5°F.) was reached in July and August, being followed then by a gradual dropping until the freeze-up.

In the distribution system of the city the temperature of the water rises somewhat in winter and falls in summer. In the heart of the city, where the pipes are large and the quantities of water used are large, the temperatures do not vary more than a degree or two from those in the city reservoir; approaching the limits of the city there is a gradual rise in winter to a temperature of about 2.5°C. (38.5°F.) and in summer a gradual drop to a minimum of about 6°C. (43°F.).

The outstanding feature respecting temperatures is that throughout those cold winters, when the cold penetrates the ground sufficiently deep to cause thick hoar frost to collect on the arch of the aqueduct above the flowing water, enough heat is radiated into the water from the ground to a little more than balance that lost at the surface during winter, and the reverse during summer to enable this water to keep at a practically constant temperature while flowing about 100 miles.

This result bears out the predictions made in the report of 1913 that 4 feet of cover would be a sufficient protection to the aqueduct, as far as frost-proofing was concerned, although it was known that in some localities frost penetrated as deep as 9 feet in cold winters; in fact, in Winnipeg, small service mains freeze sometimes in June and July, at that depth.

Prior to the completion of the aqueduct, at the suggestion of General Ruttan, formerly city engineer of Winnipeg for many years, an experiment was made in a completed portion of the aqueduct to reproduce, as well as might be, the conditions to be expected of the

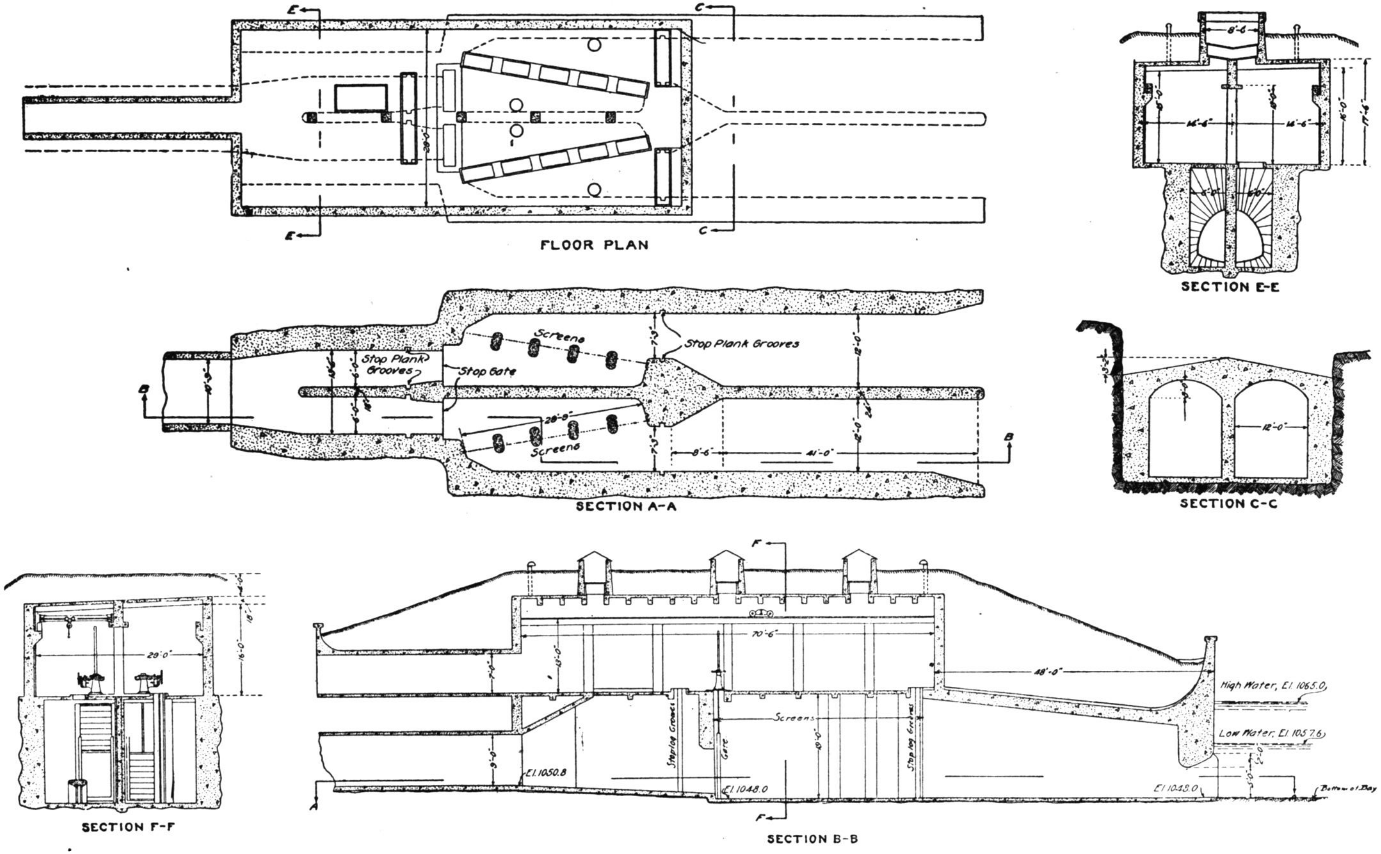

FIG. 2. THE INTAKE OF THE WINNIPEG AQUEDUCT AT INDIAN BAY

aqueduct when in operation. A section of aqueduct was chosen, running westward from the Birch River crossing, at which point water could be diverted into the auqeduct, and turned out again after passing through 3 miles of the aqueduct into a ditch at the west end of the section under test. As the aqueduct slope was too steep to control the flow by a dam at the lower end, three sand-bag dams were built in it, one at the lower end, and two others at intervals of about 4000 feet upstream. This aqueduct had the standard 4 feet of cover of sandy, loamy material. Water was regulated to enter from the river at the rate of 130,000 gallons per day, and at that rate required about a week to travel the three miles before escaping, being held back by the dams, to subject the water to aqueduct temperature for about the same time that would be required for water to flow from Lake of the Woods to Winnipeg at the minimum rate of operation, 25,000,000 gallons per day.

TABLE 1

Winter temperatures in test section of Winnipeg aqueduct, February 2, 1917

	OUTSIDE AIR	AIR IN AQUEDUCT	WATER IN RIVER	WATER IN AQUEDUCT	DEPTH OF WATER, AQUEDUCT
	degs. F.	*degs. F.*	*degs. F.*	*degs. F.*	*inches*
	−14		32		
Entrance to aqueduct		29		33	3½
First dam		36		36	40
Second dam		37		37	40
Last dam		37		37	56

The first set of temperatures was taken on February 2, 1917; they are recorded in table 1.

No ice nor frost appeared on roofs or walls of aqueduct nor on the water flowing.

From the last dam to the second, the depth of water decreased gradually from 56 to 3½ inches in about 3500 feet, and remained at 3½ inches for about 1000 feet. Between the second and first dam the depth decreased from 40 inches at the dam to 3½ inches in 2500 feet, and remained 3½ inches to the first dam, about 1500 feet.

Observations repeated in March gave practically the same results, although the outside air had risen to 32°F., and the long continued cold had penetrated the backfill over the aqueduct, causing a strip of hoar frost 2 to 6 feet wide to form to a depth of ¼ inch on the inside of the top of the aqueduct.

LINES AND GRADES FOR AQUEDUCT

Economical depth of cut. The establishment of the proper alignment for the aqueduct involved a study of costs balanced against grade lines. The preliminary profile prepared for the report of 1913 furnished the base for comparisons. The cost of an aqueduct section on any given grade varied with the position of the invert with respect to the ground level. Very shallow cuts, requiring large embankment quantities for covering the aqueduct, and very deep cuttings cost a great deal more per foot than a moderate average cut. Comparative estimates of cost, including the stripping, excavation, timber platform, concrete and backfilling showed that, for local conditions and methods of payment, an average cut of from 4 feet for the smallest to 5.5 feet for the largest section gave the minimum costs per foot of completed work, and that with a range of half a foot either side of the most economical depth there was practically no difference in cost.

After the preliminary line had been run through from Winnipeg to Shoal Lake it was found that there were three critical points through which this line had to pass, one was the west end of Snake Lake, near the Lake of the Woods, another was the crossing of Brokenhead River, where moving the line very far either side of the chosen crossing would greatly affect adversely the grade line or increase the length of the inverted syphon required to cross this valley or plain; and the last was the passing from the Brokenhead to the English River drainage basins, where there was but one low saddle to follow without swinging the line far away from the short route.

Foundation conditions. With these points fixed, the problem resolved itself into finding the most suitable alignment, for the least cost, to connect the points into a continuous line. Had the country been dry or the sub-soil the same throughout, the problem would have been much simpler, but, in places, although the natural ground level was at a favorable height, the sub-soil was so wet and soft as to be unsuitable to carry the aqueduct, requiring either, 1, a pile foundation; 2, the removal of the soft ground to a solid bottom and refilling of the trench with gravel hauled in by train and deposited in the trench in water kept standing high enough to entirely submerge the gravel until the surface was raised a few inches above the finished grade of the aqueduct (many miles of foundation had to be prepared in this way); or 3, the depression of the aqueduct sufficiently below the hydraulic grade to secure a solid bottom and the reinforcement of the concrete work so as to stand the resulting internal pressure.

As finally located there were nine of these depressed sections, as shown in table 2.

Effect of slope on cost of aqueduct. The general problem as to alignment, therefore, was one of finding the line giving a grade averaging as close to the mean slope between termini as possible without diverging so far from the shortest practicable line as to have the extra length cost more than would be saved in concrete, excavation and foundation costs by using a longer alignment on a flatter slope. Thus, with the 7-foot by 8-foot 3¾-inch aqueduct the line could be lengthened 14 per cent to avoid a cut 6 feet greater than the most economical depth of cut.

TABLE 2

Depressed sections on Winnipeg aqueduct

LOCATION	DIAMETER		LENGTH	MAXIMUM PRESSURE HEAD
	feet	*inches*	*feet*	*feet*
McPhillips St. Reservoir to Red River	4	0	11,400	*20 to 65
Red River to Deacon	5	6	49,900	20 to 50
Deacon, 658 + 60 to 900 + 30	8	0	24,140	10 to 30
Brokenhead River, 2080 + 00 to 2220 + 00	7	6	14,000	7 to 25
Whitemouth River 3383 + 77 to 3386 + 25	6	9½	248	26
Birch River, 3888 + 27.6 to 3890 + 17.5	7	3	190	20
1st Boggy River, 4098 + 80 to 4101 + 00	8	0	220	26
2nd Boggy River, 4332 + 95 to 4334 + 85	8	0	190	21
Falcon River, 5105 + 40 to 5110 + 232†	9	0	483	15 to 20
Total length			100,771	

* Tested to 40 pounds per square inch.

† Includes Venturi meter.

Except at the critical points spoken of, the entire country was so flat, so heavily covered with brush and timber, or so wet and boggy, that the eye was no guide in picking out the proper line. Levels had to be run on offsets from the preliminary lines, sometimes as far as 20 miles at right angles, along section lines that had been cut out, to get a comprehensive contour map, with contours at 1 or 2 foot intervals, to trace out thereon the best line to follow to secure good grades, good lines and avoid difficulties in other places. In addition to this, soundings and wash borings had to be put down in soft places and in swampy areas to locate the hard subsoil, and determine whether to use a foundation fill, depressed section or pile foundation to get the aqueduct through, and determine the cost.

Hydraulic gradient at east end of aqueduct. In the establishment of the grade of the aqueduct there were some novel points. The first relates to the passage of the aqueduct from Indian Bay to the valley of Boggy River, through a deep cut 9 miles long. The height of the water in Indian Bay fixed the range of levels available to govern the entrance of the water into the aqueduct, and the depth of cutting, influenced greatly by the grade of the aqueduct, controlled also the size of the section and hence the cost. Naturally a very slight slope was adopted for the long 9-mile cut, at the end of which the grade steepened considerably, and this circumstance was utilized in the following way:

The grade adopted for the 9-mile cut was 0.11 foot per 1000, and the depth of flow on this slope to discharge 85,000,000 gallons daily, through the 10 foot 9 inch by 9 foot section of aqueduct, would be 7.25 feet. The slope of the next section was 0.279 foot per 1000 feet and the depth of flow, with the 8 foot 10¼ inch by 7 foot 5⅝ inch section, 6.26 feet for 85,000,000 gallons daily discharge. By continuing the large section past the junction of the two grades and down on the steeper slope for 11,100 feet, then changing to the smaller section, the water surface at the junction of the flat and the steep slopes would drop to give a depth of flow of 4.98 feet instead of the 7.25 feet required to discharge the same quantity on the 0.11-foot grade, and the larger aqueduct would have the advantage of this additional slope without the necessity of dropping the aqueduct grade.

In other words, this expedient to secure discharging capacity made possible the placing of the invert grade through the deep 9-mile cut an average of about 10 inches higher than if the large section had ended and the smaller begun at the break of grade, and still discharge the required 85,000,000 gallons per day at low water in Indian Bay. Also, using the 10 foot 9 inch by 9 foot aqueduct had the further advantage that at high water in the lake the capacity of this large section would be about 120,000,000 gallons per day, so that at the very slight additional cost required for the larger section, this portion, the most expensive of all the aqueduct, would not require duplication for a very long time in the future. These conditions are shown in figure 3.

Provision for second aqueduct on increasing supply in future. Should, in the future, the rate of consumption pass 85,000,000 gallons, it is contemplated to install a booster pumping station to raise the lake water the few feet required, when required, and run the aqueduct section under a slight head, from nothing to about 4 feet, for the first 5 miles of its length. Provision was made in the works as built, for the junction of a second aqueduct to the existing one at the end of this extended large section, so that advantage could be taken of these conditions when the demand for water exceeds the capacity of the existing aqueduct, without interfering with the supply of water to the District.

General description of aqueduct. From this point to the site of the proposed equalizing reservoir at Deacon, about 12 miles from Winnipeg, the problem of grades was simply one of adjusting the alignment to secure that line which could be built for the least cost, in other words, equating lengths and corresponding unit costs as determined from the resulting grades and foundation difficulties to give the least cost between termini. Usually, in locating gravity aqueducts, there is little choice in selecting a route, topographical difficulties controlling and limiting the location to one practicable line. In the case of the Winnipeg aqueduct, however, the country was so flat and its general slope so uniform, that, as far as slopes were concerned, an aqueduct could have been run through on an almost straight line between Winnipeg and Shoal Lake. Such an alignment would, however, have cost from $2,000,000 to $3,000,000 more than the one finally located, although several miles shorter, as it would have consisted of long stretches of nearly level grades, joined by short stretches of steep grades, and the resulting aqueduct would have been made up of a large percentage of large sections and a small percentage of small sizes of aqueduct.

From the intake westward, to within about 17 miles of Winnipeg, a distance of 80 miles, the slope of the ground is such that a gravity aqueduct, planned to run not quite full, was used. The section is horseshoe-shaped, as will be shown later, except where depressed below the hydraulic grade. These portions are circular and reinforced for internal pressures. The largest gravity flow section is that through the summit cut, above described, which is 10 feet 9 inches wide and 9 feet high on a slope of 0.58 foot per mile; the

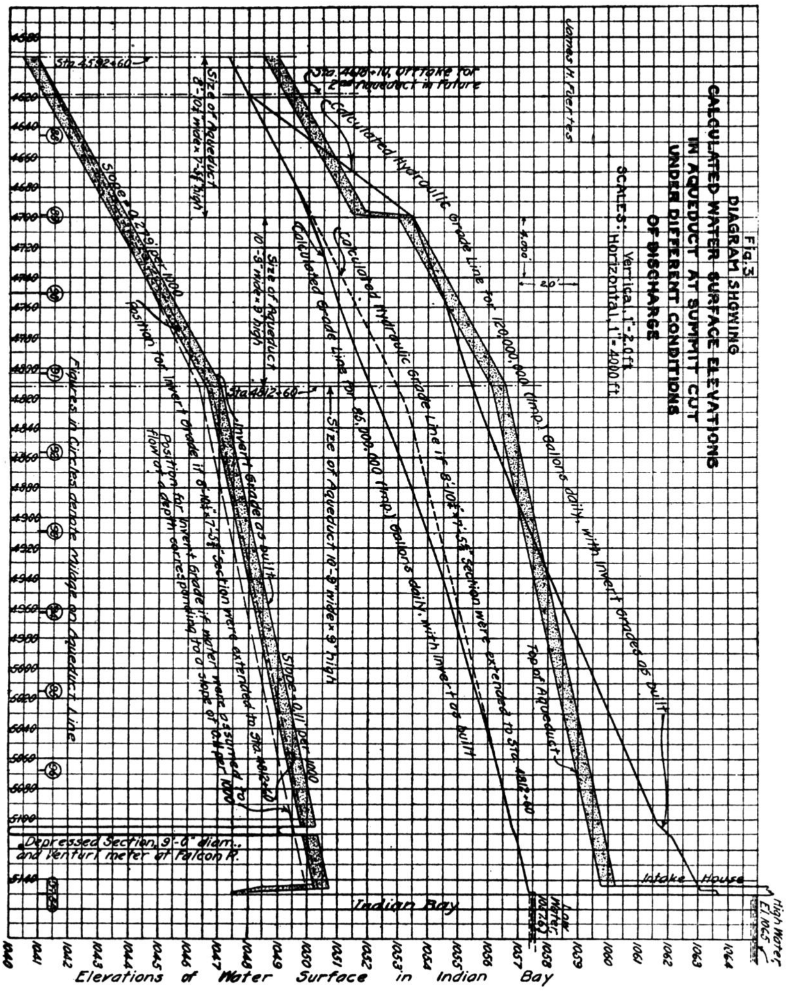

Fig. 3
DIAGRAM SHOWING
CALCULATED WATER SURFACE ELEVATIONS
IN AQUEDUCT AT SUMMIT CUT
UNDER DIFFERENT CONDITIONS
OF DISCHARGE

SCALES: Vertical, 1" = 2.0 ft.
Horizontal, 1" = 4000 ft.

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smallest is between miles 23½ and 32½, which is on a slope of 1.537 feet per 1000 feet (8.11 feet per mile, the steepest slope on the whole line) and is 6 feet 4¾ inches wide and 5 feet 4¾ inches high. From Mile 17 westward toward Winnipeg the ground falls off in elevation so that to deliver water into the Winnipeg reservoir, the aqueduct must run under pressure, and here, again, enter some interesting hydraulic problems. All the aqueduct, above described, that is from Mile 17 eastward for 80 miles, has a discharging capacity of 85,000,000 gallons daily, a quantity far in excess of the needs of Winnipeg for many years.

HYDRAULIC GRADIENTS AT WINNIPEG END OF AQUEDUCT

General conditions. The plans as worked out for this western end are based on construction details such that quantities up to 28,500,000 gallons daily may be delivered by gravity through the system into the McPhillips Street reservoir in Winnipeg at elevation 769.5, which is high water level in that reservoir. That is, from the western end of the gravity flow aqueduct at Mile 17 to Deacon, at Mile 13, there is a depressed section, having a diameter of 8 feet, followed by 49,900 feet of 5 foot 6 inch lock-joint pipe, reaching to Red River in Winnipeg; then a 5 foot diameter cast-iron-lined inverted siphon under the river, and a 4-foot lock-joint reinforced concrete pipe 11,400 feet long through the streets of Winnipeg to the McPhillips Street reservoir.

As a part of the system, but not for immediate construction, it is planned to provide at Deacon, where the 8-foot and 5½-foot concrete pipe sections join, a reservoir holding 250,000,000 gallons to serve as an equalizing basin and as a safeguard against temporary interruption of flow from Shoal Lake, should this be necessary to permit repairs or cleaning of the aqueduct. The natural ground level at the site chosen for this large reservoir is at an average elevation of about 774.5 and the water level will vary in service from 791.0 to 797; in other words, it will stand from 16.5 to 22.5 feet above the level of the surrounding prairie. The higher level represents the condition when about 20,000,000 gallons daily is going by gravity to the McPhillips Street reservoir, and the lower when about 115,000,000 gallons daily is leaving the reservoir and is distributed to the various communities forming the water district, 35,000,000 going to the bank of Red River by gravity and being boosted thence through the 48-inch pipe line to the McPhillips Street reservoir, and the balance being distributed to the city and its associated communities from other points.

In order properly to consider the hydraulic problems involved in the west end of the aqueduct, from the Deacon Reservoir to Winnipeg, it will be necessary to say a word about the quantities of water required by the different communities forming the district, the rates at which it must be supplied, and the obligations of the District to its component parts in the matter of purveying the water.

Essential conditions in Greater Winnipeg water district act. 1. The corporation is required to furnish water "in bulk" to the several communities forming the district. The context of the Act discloses that the words "in bulk" mean quantities of water at ground level rather than at city street main pressures. In other words, while the District is not authorized to pump water into the street mains of any of the municipalities, it is commanded to furnish water to any one or all, in such quantities as they may severally or collectively need from time to time, by gravity if possible, and, if not, then by pumping to sufficient head to cause the water to flow from the nearest convenient point on the District's main conduit to the nearest point at the corporate limits of the municipality in question; however, this Act specifically states that the main conduit shall terminate at a point adjacent to the McPhillips Street Reservoir in Winnipeg.

2. Any municipality having water conveyed to it from the District's main conduit through a special connection and branch conduit, shall pay the District annually half the interest, sinking fund, maintenance and operation costs in respect of plant and construction required to convey water from the main conduit of the corporation to the municipality in question.

3. Any municipality having a system of water mains may deliver water through these, in bulk, to another municipality.

4. The corporation is not required to build a main to deliver water to any municipality when another municipality is willing and able to deliver it an adequate supply, but may be required to later build such a main, if the aforesaid arrangement is terminated.

5. The corporation may bear in whole, or in part, the cost of extending the existing mains of any municipality to the boundary of another to avoid the cost of direct mains from its works.

Population growth and water consumption requirements. As a basis for the plan for distributing the water to the different municipalities therefore, the quantities of water required by each, and the rates at which it may be needed, now or in the future, are of vital importance.

In tables 3 and 4 the populations are based on the actual populations at the time of the formation of the District, extended in proportion to their areas at certain assumed increasing densities of population per acre, based on a study of the rates of increase in the older communities.

TABLE 3

Estimated populations and rates of average daily water consumption in millions of gallons in the communities forming the Greater Winnipeg water district

COMMUNITY	1920		1930		1940		1950	
	Population	Consumption	Population	Consumption	Population	Consumption	Population	Consumption
Winnipeg......	218,000	16.13	305,000	25.31	397,000	36.52	490,000	49.00
Fort Gary.....	8,000	0.59	17,000	1.41	29,000	2.67	42,000	4.20
Assiniboia.....	13,000	0.96	27,000	2.24	43,000	3.96	61,000	6.10
Kildonan......	6,000	0.44	13,000	1.08	22,000	2.02	35,000	3.30
St. Boniface...	20,000	1.48	45,000	3.74	85,000	7.82	134,000	13.40
St. Vital.......	4,000	0.30	8,000	0.66	13,000	1.20	20,000	2.00
Transcona.....	7,000	0.52	17,000	1.41	30,000	2.76	45,000	4.50

TABLE 4

Estimated population and rates of daily water consumption in millions of gallons in the entire Greater Winnipeg water district

YEAR	POPULATION	AVERAGE DAILY RATE	MAXIMUM DAILY RATE	MAXIMUM HOURLY RATE	EXTREME HOURLY RATE INCLUDING FIRE SERVICE	REASONABLE MAXIMUM HOURLY RATE INCLUDING FIRE SERVICE
1915	207,000	13.65	19.10	31.52	44.12	37.50
1920	276,000	20.49	28.59	45.15	60.15	51.13
1930	432,000	35.85	50.18	75.24	93.24	80.25
1940	619,000	56.95	79.73	113.17	133.57	117.54
1950	825,000	82.50	115.50	159.23	181.43	159.23

The necessity of considering fire drafts in these estimates arises from the fact that, although the city now has a high pressure fire service, in future the fire pumping engines will be supplied with water from the new aqueduct, and the aqueduct system will respond to the demands as occurring, the same as though the water were to be pumped from the McPhillips Street reservoir.

The foregoing estimates were intended for the purpose, only, of forming a basis for the proportioning of the sizes and capacities of the future water supply works the District will have to provide. They may, in individual districts, prove to be more or less inaccurate, but for the purposes intended exactness is not essential, for very considerable variations from the true conditions will but slightly affect the total and will have only a negligible effect on the actual dimensions of the works. They are to be regarded more as aids in fixing upon the ultimate limits then as representing actually the absolute variations to be expected from year to year.

Effect of rate of draft on capacities of pipe line, force mains and pumps. The rate of use of water by a community varies from hour to hour and from day to day; and the actual demands, as may be seen in table 4, may, in the Winnipeg District, amount to two and a half to three times the average daily rate of consumption. To allow for such fluctuations, either the aqueduct must deliver the water as fast as required at any moment, and its actual effective capacity be only from 40 to 33 per cent of its full capacity, or a storage reservoir must be provided near town to supply the water required by sudden heavy drafts.

Deacon reservoir a necessity soon. Where long aqueducts are in service it is more practicable to build large storage reservoirs than to build the aqueduct large enough to accommodate the maximum possible draft for short times, the purposes of these large reservoirs being to allow a full and large supply in response to sudden heavy drafts without necessitating sudden changes in velocities in the pipe line, and to store excess water, not needed during hours of light draft, for use later during heavy drafts.

In an aqueduct conveying water by gravity flow under limited and comparatively low heads, it is not practicable to quickly speed up or check down the velocity of the water in the conduit, as it would be in a closed force main; hence a considerable amount of storage is necessary to permit satisfactory operating conditions.

The difference in rate between average rates of draft and maximum rates is so great, and high rates may prevail so long, that while a day's supply of water in storage is sufficient to balance extremes of draft when the supply can come to the reservoir at a rate equal to the daily maximum rate of draft, it would take several weeks of storage capacity if the water entered the reservoir at the average daily rate of draft to afford the same balancing effect. For this reason it is quite usual, where the supply can come to the reservoir at the maximum daily rate of draft, to provide at least one day's storage for carrying the peak drafts.

As above stated, the original plans provided for a reservoir of 250,000,000 gallons capacity to be built at Deacon, in line with the above policy. The original plans, however, provided for discharging the water from Deacon to Winnipeg by gravity, up to a limit of 25,000,000 gallons per day, and pumping the water from Deacon to Winnipeg when the consumption shall have exceeded 25,000,000 gallons per day. For this purpose it was provided that the force main from Deacon to Winnipeg should be a 5-foot riveted steel pipe, as far as Red River, then a 5-foot cast-iron-lined tunnel under Red River and then a 4-foot cast-iron pipe from Red River to the McPhillips Street Reservoir. The pumping station at Deacon was intended to pump the water at sufficient pressure to supply the District through the existing street mains in the different municipalities.

Effect of delivery "in bulk" on designs. A reading of the Act, however, discloses that there was no warrant for the delivery of the water, other than "in bulk," and this led to a change in the plans, whereby the water would be delivered in the future, to the different municipalities at ground level, to be repumped by each municipality in accordance with its own needs. This change will limit the variation in rate of delivery from Deacon to the District between the minimum rate and maximum daily rate instead of the minimum rate and maximum hourly rate, which latter would be the case if the pumping were done at Deacon.

The Deacon Reservoir will, therefore, be used for the same purposes as in the original plan, but may be of smaller capacity than originally intended, owing to the fact that part of the original capacity provided, namely, that part required to balance the hourly variations in pump drafts, must be provided nearer the Red River.

For some years there will be no urgent necessity for the large reservoir. Cleaning of the aqueduct will not be needed for several years, and it will be many years hence before the aqueduct between Deacon and Shoal Lake will be called on to deliver water up to its maximum discharging capacity, even at maximum rates of consumption in the District. Nevertheless, with but one aqueduct nearly 100 miles long, it would be prudent to get the reservoir built at an early date.

Taking into account the changes in operating conditions due to the adoption of the plan of delivering the water to the district at ground level instead of pumping it into the mains, the capacity of the Deacon Reservoir can be reduced to 200,000,000 gallons, provisions being made for supplemental storage in the future at a point not far from Red River, sufficient to take care of fluctuations in the hourly rate of pumping, when required, or about 60,000,000 gallons, as will be described later on.

Supply conditions between Deacon and Red River. The plan adopted was to provide at the end of the 8-foot circular aqueduct at Deacon for a connection to the Deacon Reservoir, when built, with a by-pass direct to the 5½-foot concrete pipe from Deacon to Red River, connections being also provided so that water may be later taken into the 5½-foot concrete pipe from either half of the proposed reservoir. All the water will, whether going to the reservoir and thence to the 5½-foot pipe line leading to the Red River or being by-passed directly to the 5½-foot line, pass through a Venturi meter built into the line. At the east bank of the Red River the 5½-foot concrete pipe ends in a surge tank having a 48-inch cast-iron connection to the shaft at the west end of the tunnel under Red River and also a 42-inch cast-iron connection for the suction pipes of booster pumps to be used, later, when necessary, to force the water to McPhillips Street in the quantities required. An over-flow from the surge tank discharging through a 36-inch cast-iron pipe to the river will give relief so that the pressures in the 5½-foot concrete pipe will not exceed certain predetermined amounts when the water may surge therein, due to the starting or stopping of the booster pumps.

There is no valve between the surge tank and the aqueduct, so that by no accident of operation can this connection be closed. The connection between the surge tank and the tunnel under Red River, however, is provided with a valve in the west shaft of the tunnel to enable the surge tank to be cut off; and a valved connection to the tunnel will permit the water taken by the booster pumps from the surge tank to be forced through the tunnel and thence to McPhillips Street, when more water is needed there than will flow there by gravity through the aqueduct from Deacon.

Having thus outlined, in a general way, the conditions to be fulfilled, that is, to deliver to the McPhillips Street reservoir as much water as practicable by gravity and then boost over from Red River surge tank as much as required, bearing in mind the requirements of each of the municipalities forming the District, the problem became one of fixing upon the proper sizes for the pipe lines and tunnel, the capacities of the booster pumps and the dimensions and details of the surge tank, and to fix upon a plan for the distribution of the water to the different municipalities, in bulk in proportion to their needs, present and future, and to have the works when built such that they could economically be adapted to whatever final plan of future development the District might adopt.

Supply for Winnipeg and its dependents. The city of Winnipeg proper, which, before the introduction of Shoal Lake water supplied Assiniboia, Kildonan and Fort Garry, has at McPhillips Street a 25,000,000-gallon covered reservoir, together with a pumping station used for pumping the water from this reservoir into the city street mains direct. There are no high level distribution reservoirs or standpipes in the system, so that the pumps must respond to all changes of pressure corresponding to a varying rate of draft. This reservoir is large enough to serve until the population shall have grown enough to require an average of about 25,000,000 gallons of water daily, corresponding, for local conditions, to a maximum daily rate of 35,000,000 gallons.

This quantity, therefore (35,000,000 gallons daily), would represent the capacity required in the pipe line from Red River to McPhillips Street to utilize the reservoir to its full practicable capacity. With this condition, the maximum actual rate of pumping of water out of McPhillips Street reservoir might be as great as 60,000,000 gallons daily for an hour or two, the extra quantity of water for these short periods being furnished from the storage in the reservoir. As will be explained later, however, the pipe line from Red River to McPhillips Street reservoir must be able to deliver water at the rate of 50,000,000 gallons per day, as that is the rated capacity chosen for the booster pump.

The McPhillips Street pumping station and reservoir are located in the city of Winnipeg and occupy all the land available for the purpose, so that any increase in reservoir capacity beyond that now available, must, on account of the value of land thereabouts, be had elsewhere. The pumps at McPhillips Street could be increased in number and capacity, and a pumping capacity in excess of 25,000,000 gallons daily could be maintained there by putting the booster pumps at Red River in use to boost over the extra quantity, beyond that which will flow over by gravity, required for peak loads. Careful studies of this question, however, showed that it would be preferable to definitely limit the capacity of the McPhillips Street station to an average 25,000,000 gallons daily, and establish another pumping station on the east side of Red River for furnishing the extra water in future, through new force mains, when more than 25,000,000 gallons daily should be required in the district now served by the McPhillips Street plant. This plan is simple and has many advantages, among which the greatest is that without sacrificing any of the work to be done under it, at the present time, it could be changed at any time to fit in with any of the other practicable schemes of development. The dimensions of the works, as built, therefore, are determined in accordance with this plan, which involved the least expenditure for the district, both for construction and operation.

Program for development. The practical effect of these assumptions on the design of the works is seen from figure 4, which shows the hydraulic gradient under various conditions of draft.

At the right hand side of the diagram, at station 900, is shown the western end of the last gravity flow aqueduct section, which is 8 feet 9 inches wide and 7 feet 4¾ inches high, with its invert at elevation 792.0. The top of the arch would therefore be at elevation 799.40. In order not to have any upward pressure on this arched section the elevation of the hydraulic grade was fixed at 797.5 for the maximum rates of draft at the different critical rates.

A number of studies of different sizes of pipes, with the resulting grades for discharging the water, showed that for the materials to be used a 5½-foot reinforced concrete pipe from Deacon to Red River, and a 48-inch reinforced concrete pipe from Red River to McPhillips Street reservoir, would be most economical for immediate construction and would take care of the consumption until about 1932, at the rates of increase in use of water as given in Tables 3 and 4. This program would require, after that date, the laying of another 5½-foot conduit from Deacon to St. Boniface with provisions at that place for pumping the extra water directly into the street mains of the municipalities both sides of Red River.

At St. Boniface will be required a storage reservoir or pump-well holding about 10,000,000 gallons at the start, to be increased to about 60,000,000 gallons capacity eventually, but by degrees, to equalize the flow through the aqueduct during times of excessive pumping rates; and, for proper service, the smaller capacity of storage, the requisite pumps and the connection from the first 5½-foot conduit should be ready by about 1926. This part of the ultimate plant, however, and the large reservoir at Deacon, are not yet a necessity and have not been built or provided for, except as to connections with the 5½-foot conduit, which are all in place and ready when needed. At the St. Boniface pumping station the water will be pumped to street main pressure and the cost of the station and the operation be borne by the municipalities benefited, rather than by the District, which has fulfilled its obligation by bringing the water, in bulk, to all its component municipalities.

It will be observed, from figure 4, that the quantity of water to be delivered to the McPhillips Street reservoir is limited to an average of 25,000,000 gallons daily (35,000,000 gallons maximum rate) and that quantities up to 28,500,000 gallons daily can be delivered into that reservoir by gravity. This latter figure would correspond to an average daily consumption of a little over 20,000,000 gallons. The hydraulic grades shown in each case are for the maximum daily rates of consumption.

It will be observed, also, that the head on the 5½ foot pipe may vary from 12 to about 47 feet, the greatest head being for the delivery of about 20,000,000 gallons per day to McPhillips Street by gravity, and the least for the maximum consumption and pumpage at St. Boniface, years hence. After about 1926 the conditions of discharge to McPhillips Street will remain practically the same as to routine operation; that is, an average of 25,000,000 gallons daily will be sent to McPhillips Street, part of it by gravity and the remainder by booster pumping. The lift on the booster pumps in 1926 will be about 40 feet; this will increase gradually to a maximum of 61.5 feet by 1942, when two 5½-foot conduits are in use and the total average consumption in the district about 100,000,000 gallons daily, nearly 50 per cent in excess of the average daily consumption that can be taken care of by the present gravity flow aqueduct between Deacon and Shoal Lake.

Protection of line from strains due to surging. An inspection of figure 4 in connection with the variations in rates of draft between minimum and maximum rates, as given in Table 4, shows that provisions had to be made to protect the portion of the aqueduct between station 900 and Winnipeg against excess pressures due to surging of the water in response to changes in velocities of flow caused by more or less sudden variations in draft, particularly when the booster pumps at Red River would be thrown in or out of ser-

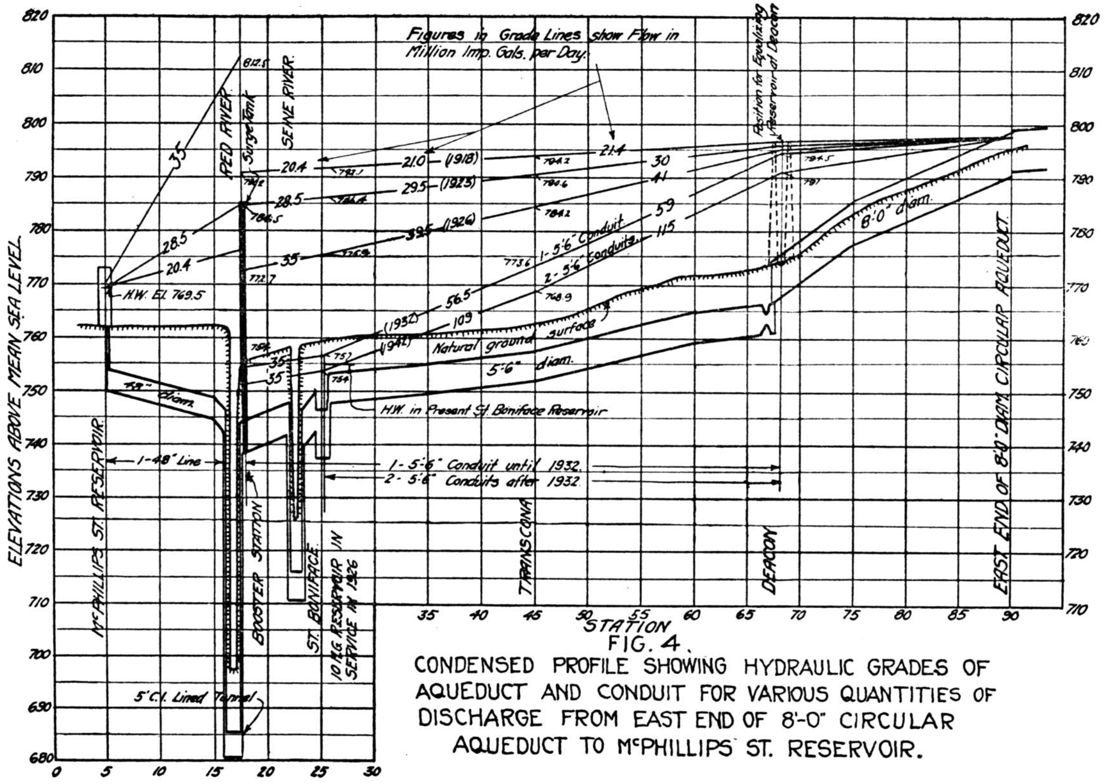


FIG. 4.

CONDENSED PROFILE SHOWING HYDRAULIC GRADES OF AQUEDUCT AND CONDUIT FOR VARIOUS QUANTITIES OF DISCHARGE FROM EAST END OF 8'-0" CIRCULAR AQUEDUCT TO McPHILLIPS ST. RESERVOIR.

vice. These pumps are to be installed with a capacity of 50,000,000 gallons daily, so that pumping can be done at a rather high rate for short times daily rather than be continuous. The storage capacity of McPhillips Street reservoir makes this feasible, and simplifies the pumping machinery.

In order to limit the excess pressures from the above causes, the aqueduct has been connected to a stand pipe, with an overflow with its lip at elevation 785.5, a height somewhat greater than required for the delivery of 28,500,000 gallons daily into McPhillips Street reservoir by gravity, and an overflow has been placed on the gravity flow section at station 900+30 with its lip at elevation 797.60. The last mentioned overflow will protect the gravity flow aqueduct to the east of the 8-foot diameter circular pressure section, and the two overflows, combined, will prevent excessive pressures in the portion between station 900 and Red River.

The surge tank. The surge tank is a reinforced concrete structure, circular in plan, containing a central well, into which the aqueduct discharges, 25 feet in diameter with its top edge at elevation 785.5. This well is inside of and concentric with a second concrete well 32 feet 6 inches inside diameter, with its top carried up to support a reinforced concrete roof, the under side of which is at elevation 294.75, or 9 feet 8 inches higher than the overflow lip.

Both of these circular wells are carried on a circular concrete base some 8 feet in thickness, resting in a grillage of reinforced concrete and heavy steel beams encased in concrete, carried on eight concrete piers extending to solid rock about 34 feet below the inside bottom of the surge tank proper. Surrounding the whole structure, and separated from it by an annular space 2 feet 9 inches wide at the bottom, reducing to 9½ inches at the top of the structure, is a brick facing with stone base, belt courses and cornice. This design is primarily due to the necessity of conserving the heat of the incoming water in order to prevent the freezing up of the first annular space when overflows take place in winter weather. The water overflowing the inner circular well escapes to the river through a 36-inch cast iron pipe line.

The original calculations showed that a tank 20 feet in diameter would be large enough to limit the extreme downward surge to a practicable limit under the assumption of a discharge at the rate of 18,900,000 gallons daily into the tank being suddenly increased to a discharge of 51,000,000 gallons daily by throwing in a booster pump of that capacity. This proceeding would increase the velocity in the $5\frac{1}{2}$-foot pipe line from 1.47 feet per second, prevailing before the booster was started, to 4 feet per second when full discharge was established. An analysis by a process of arithmetic integration at 20 second intervals of time indicated that the lowest dip of the surge reached an elevation of 751.1, and then started to rise again, at about 6 minutes 20 seconds after starting the pumps. The low point was about 2.3 feet below the final level for a continuous discharge at a velocity of 4 feet per second. The maximum discharging velocity, 4.1 feet per second, was reached in about 8 minutes 20 seconds after starting the pumps. The upward surge from the low point was not followed out in the analysis; neither was the surge from sudden shutting down of the pumps, which would be the maximum to be expected, as these would be entirely checked by the spilling of the water over the overflow lip.

The original plan was for the use of 5-foot steel pipe line from Deacon to Red River and a 4-foot cast-iron pipe from Red River to McPhillips Street Reservoir, and to pump the water from Deacon to Winnipeg at service pressure. As has been pointed out, this plan could not be followed out on account of the conditions in the District's act of incorporation, although, of course, the pipes could be used as recommended. As the new conditions did not require such strong pipes studies were made which showed that reinforced concrete pipes could be substituted for those originally proposed, and a new plan of operation be evolved which would reduce the cost of construction and operation while at the same time providing for the delivery of larger quantities of water to the McPhillips Street Reservoir by gravity. This change was submitted to Dr. Hering and Mr. Stearns, of the original Board of Consulting Engineers, and had their approval.

General description of completed works. Reviewing the subject, it will be seen that the works constructed include the following:

1. Intake at Indian Bay.
2. Venturi meter in the depressed section under Falcon River, about a mile from the intake.
3. Gravity-flow aqueduct on various grades, capacity 85,000,000 miles, with a connection at Station 4618+10 (Mile 87.5) for a second aqueduct in the future.
4. Eight-foot circular reinforced concrete aqueduct, built in trench, from Station 900 to Deacon (Station 678+72.5), a distance of

22,127.5 feet (4.2 miles) running in service under heads of from 10 to 30 feet; capacity 85,000,000 gallons daily.

5. Venturi meter in 8-foot pipe line, then offtake pipe to Deacon Reservoir, then 5-foot shut-off gate in aqueduct, then intake of 5½-foot line from Deacon Reservoir, then Venturi meter on 5½-foot line. These two meters, the two offtakes and the shut-off gate in the 5½-foot conduit are all in a stretch of 294.5 feet of aqueduct, measured along its center line.

6. Then a 5½-foot reinforced concrete pipe from Deacon to the surge tank at Red River; total length 49,900 feet, delivering water to the surge tank at Red River at the maximum practicable rate of 59,000,000 gallons daily from Station 900 to the 36-inch outlet connection for Transcona, 56,500,000 gallons daily to the 36-inch outlet connection for St. Boniface and St. Vital, and 35,000,000 gallons daily from St. Boniface to the surge tank at Red River. A 24-inch outlet, also, is provided at Archibald Street, for the Elmwood District and East Kildonan.

7. The Surge Tank at Red River.

8. Tunnel under Red River, with connections at top of the east shaft for the booster pump which will, later, pump water from the surge tank to the McPhillips Street Reservoir in quantities up to 35,000,000 gallons per day, but at rates up to 50,000,000 gallons per day.

9. A 36-inch connection at top of west shaft of tunnel for a future supply of water to the high-pressure fire pumps in Victoria Park on the bank of Red River, where the west shaft of the tunnel is located, and a 24-inch connection for Fort Garry.

10. A 48-inch reinforced concrete pipe from the Red River tunnel to McPhillips Street Reservoir in Winnipeg, with a 34-inch outlet at King Street for Kildonan and a 24-inch outlet for Assiniboia at Arlington Street. In this pipe line a Venturi meter has been placed just before the pipe reaches McPhillips Street Reservoir.

The capacity of the 48-inch pipe line and 5½-foot line, acting together to deliver water by gravity to the McPhillips Street reservoir, is 28,500,000 gallons per day, at the maximum rate of consumption, which will be reached when the total consumption from this center approximates an average rate of a little over 20,000,000 gallons per day. When that time arrives the booster pumps at Red River will be required and quantities representing average daily rates up to 15,000,000 gallons, above the 20,000,000 gallons, will have to be pumped to McPhillips Street at rates approximating 50,000,000 gallons per day, to keep the reservoir replenished. As has been stated above, when more than 25,000,000 gallons daily is required in Winnipeg, Kildonan, Assiniboia and Fort Garry, the additional quantity is to be pumped from a proposed new reservoir to be built at St. Boniface.

Stop-plank chambers and overflows, as well as blow-offs, have been put at all the principal river crossings, and at the end of the gravity flow aqueduct at Station 900+30, in addition to the large gates in the intake structure, for the purpose of regulating the rate of flow in the aqueduct, as required, and to prevent upward pressure on the roof or arch of the aqueduct in case of accident of any kind.

DESIGN OF AQUEDUCT SECTIONS

Arch designs. The design of the aqueduct sections was influenced by many considerations, among which the most important were:

1. Solidity of foundation.
2. Economy in the use of materials.
3. Weight and character of material suitable for backfilling over aqueduct.
4. Character and quality of the soil as to chemical characteristics and amount of moisture.
5. Depth of penetration of frost in different soils.
6. Elevation of permanent ground water level relative to invert of aqueduct, as affecting the tendency to float, and to change the form of the section after construction.
7. Practicable construction methods to secure the expedition of the work.
8. Range of temperatures of the water.
9. Character of soil as to relative amounts of clay, silica and organic matter in its composition.

The problems of arch design and invert design were influenced by quite different factors. Any arch design, to be strong enough to resist cracking, must be provided with practically unyielding foundations. In the larger Winnipeg arches a deflection of the invert by so little as $\frac{1}{40}$ inch, would crack it, and then the spreading of the feet of the arch following the further flattening of the invert would crack the arch along the center of the roof. Making the arches twice or perhaps even three times the thickness required for rigid foundations would not have prevented arch cracks under uneven settlement of the footings of the arch. The principle was therefore adopted to design the arches on the assumption of rigid foundations, and secure these, or as near these as practicable under local conditions.

The form of the arch and the thickness of the arch ring were based, in general, on a backfill load 4 feet deep over the top of the arch weighing 100 pounds per cubic foot, the width of the backfill, on top, being in all cases equal to the width of the aqueduct, inside, with a minimum of 8 feet, and side slopes for embankments of 1 vertical on 1¾-horizontal. The concrete was assumed to weigh 140 pounds per cubic foot. The stress diagrams for the arches were determined from the above loads, the earth pressure directions varying gradually from vertical at the top, in the center, to a slope towards the aqueduct of 20 degrees from the vertical, at the bottom of the aqueduct, the concrete weights acting vertically downward, the water pressure within the aqueduct, for various depths of flow, acting radially outward, normal to the aqueduct faces, and ground water pressures, when expected, acting normal to the outside surface and inward toward the aqueduct. The limiting lines for the inside and outside faces for the arches were determined from these stress lines by so adjusting these that for all conditions of loading the resultant lines of force would fall within the middle third of the section. The satisfying of this condition provides that there can be no tension in any portion of the concrete section, and therefore no tendency toward cracking of the arch.

The stresses in the B section of the aqueduct, which was used for a little over 6¼ miles, between Mile 17 and Mile 23, 10 miles, from Mile 77 to Mile 87, with the exception of two inverted siphons included in the distance, with a combined length of 410 feet, are shown in figure 5.

Under only two conditions did there appear to be a necessity for reinforcing the arch, and these were under road and railroad crossings, and where the aqueduct crosses the Brokenhead slough, where the backfill material was practically all vegetable matter weighing only about 50 pounds per cubic foot. The internal outward pressure of the water flowing through the aqueduct here would tend to produce deformation of the structure, or lifting of the arch off the invert; and reinforcing steel was necessary to tie the structure together as a unit and counteract the bending tendency in the sides of the arch. The stresses in this section are shown in figure 6.

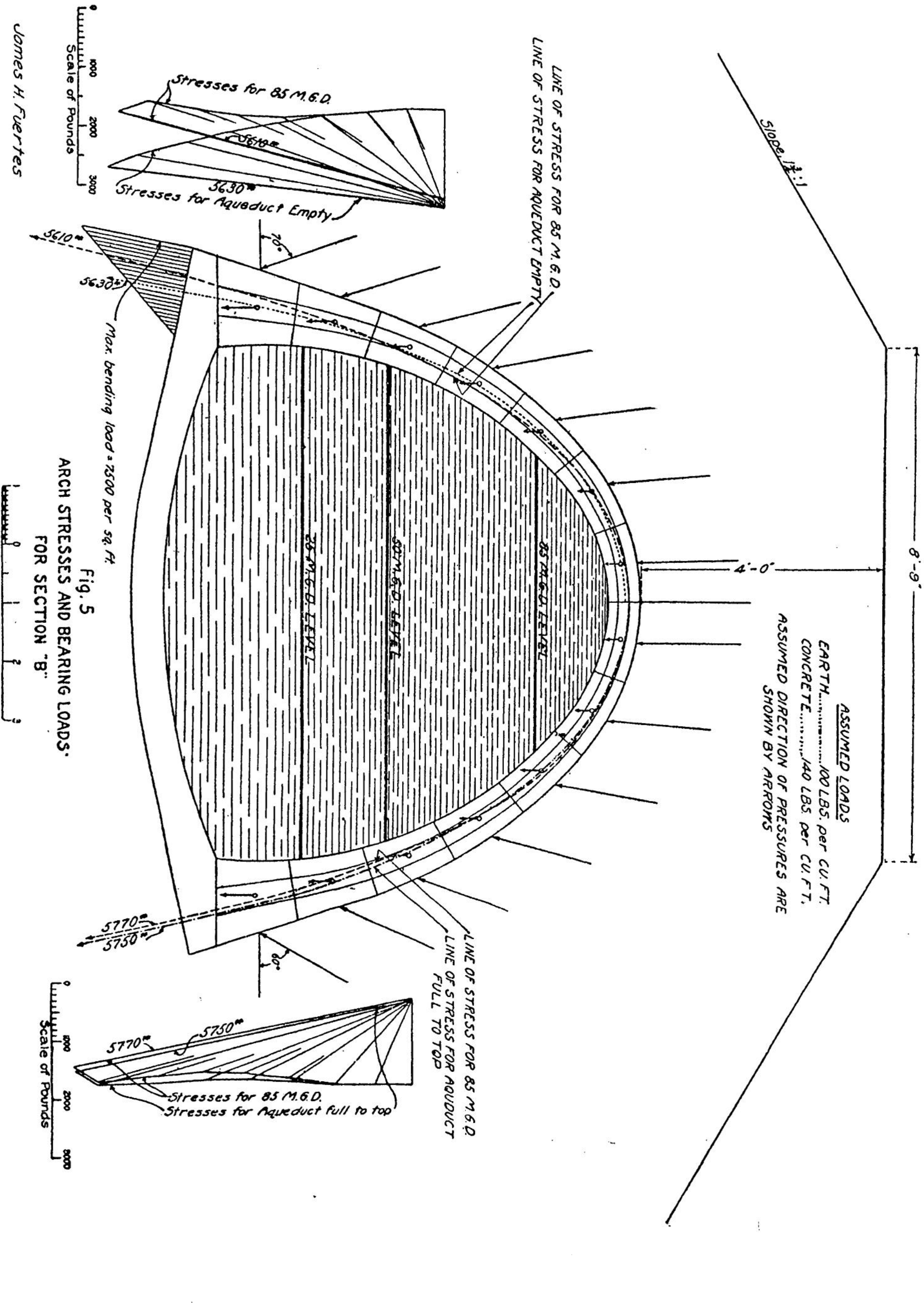

Slope 1½:1
8'-9"
4'-0"
ASSUMED LOADS
EARTH..........100 LBS. per CU. FT.
CONCRETE..........140 LBS. per CU. FT.
ASSUMED DIRECTION OF PRESSURES ARE SHOWN BY ARROWS
LINE OF STRESS FOR 85 M.G.D.
LINE OF STRESS FOR AQUEDUCT EMPTY
LINE OF STRESS FOR 85 M.G.D.
LINE OF STRESS FOR AQUDUCT FULL TO TOP
85 M.G.D. LEVEL
50 M.G.D. LEVEL
25 M.G.D. LEVEL
70°
60°
5610#
5630#
5770#
5750#
Max. bending load = 7500 per sq. ft.
Stresses for 85 M.G.D.
5610#
3630#
Stresses for Aqueduct Empty
0
1000
2000
3000
Scale of Pounds
5770#
5750#
Stresses for 85 M.G.D.
Stresses for Aqueduct full to top
0
1000
2000
3000
Scale of Pounds
0
1
2
3
Scale of Feet
James H. Fuertes

Fig. 5
ARCH STRESSES AND BEARING LOADS FOR SECTION "B"

As to the possible depth of penetration of the frost, in aqueduct embankments, or its action or effect, some rather surprising facts were noted during the winter of 1915–1916 in investigating this matter. In the prairie section, for instance, about mile 23, it was found, in one place where frost had gone down about 4 feet on the prairie, it was of less depth on the top and side slopes of the embankment over the aqueduct, which, here, was in a rather shallow cut. On digging a trench crosswise of the aqueduct, to examine the condition of the embankment, it was found that the linear expansion, parallel to the surface of the side slopes and tops of the embankment, due to the freezing of the clay soil, had caused the frozen back-fill to rise up where passing over the arch, forming a cavity between the arch and the embankment (supposed to be resting on it). This cavity was about 5 feet wide across the aqueduct, tapering from nothing at the sides to over 6 inches high in the center over the arch.

In another place, in the Brokenhead slough, at Station 2259+33, where the aqueduct is in a 7-foot cut, the frost penetrated the prairie about 4 feet, the bottom of the frozen ground extending horizontally through the backfill and against the arch, where it had frozen fast to the concrete and produced strains, the effects of which were shown in the cracking of the arch on both sides, and its being lifted vertically about ¼ inch; after frost left the ground the arch settled down again in place to its original position. The invert did not crack; the cracks in the side walls were parallel to and 3½ feet above the invert.

These effects were produced, however, before the backfill was entirely completed, and while the aqueduct was empty, during a very cold winter. Practically no expense was necessitated for repairs.

Temperature in aqueduct. Taken as a whole the depth of backfill chosen was sufficient, but not too much. Continuous observations throughout the whole length of backfilled aqueduct during the cold winter referred to showed that the temperature of the air within the aqueduct, except in the immediate neighborhood of open manholes or connections, remained around 35° to 37°F. until the advent of warm weather.

In most places frost did not penetrate to the arch until spring, and then showed in a band of hoar frost of varying widths in different places, sometimes being only a foot or so and sometimes extending down to within a foot of two of the bottom of the arch on each side. The muskeg covering was quite efficient as frost protection,

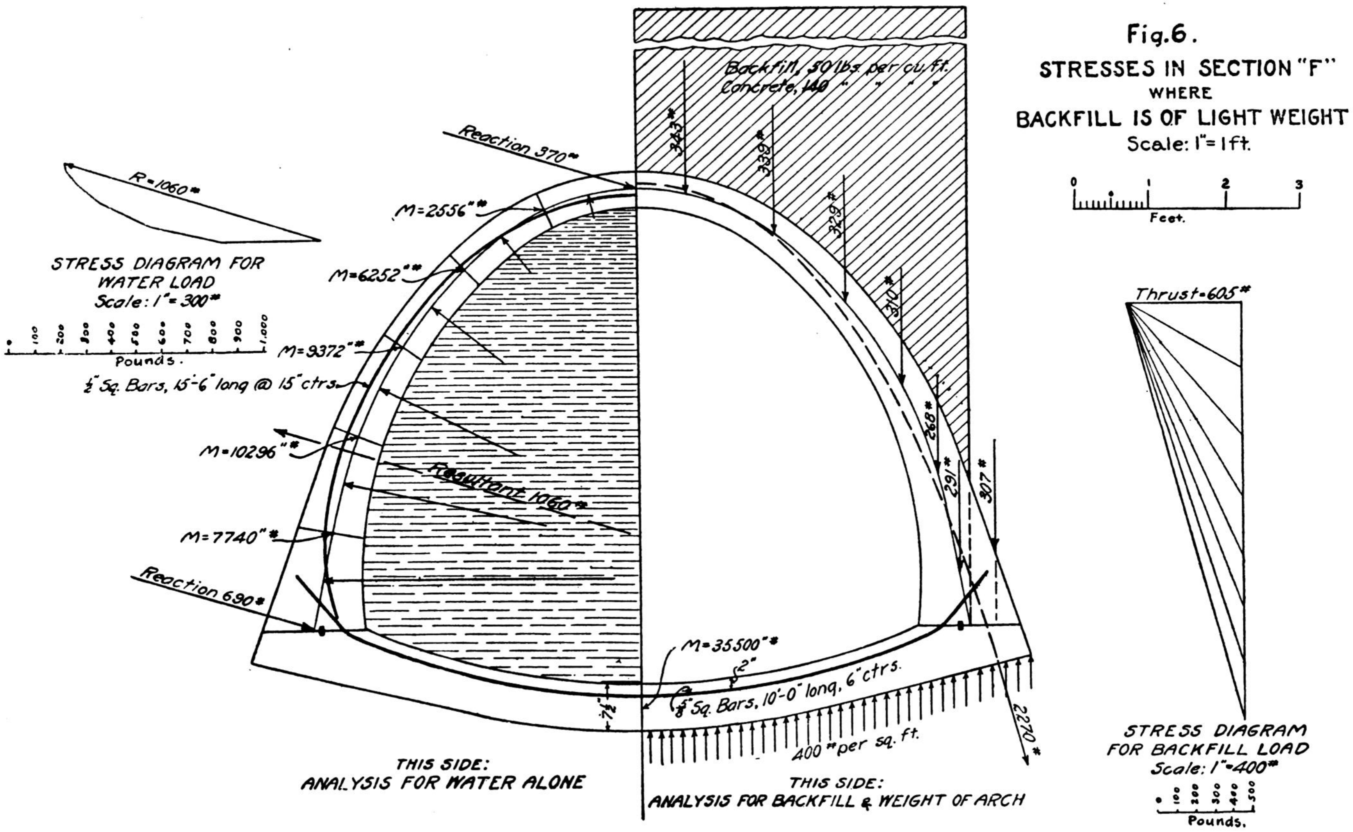


Fig. 6.
STRESSES IN SECTION "F"
WHERE
BACKFILL IS OF LIGHT WEIGHT
Scale: 1"=1ft.

but required a covering of earth or gravel as protection against fire. Frost penetrated most deeply and most quickly through moist, densely consolidated clay, less deeply and more slowly through sand and gravel, and least extensively and most slowly in the muskeg covering after it had been in place for some time and had had a chance to dry out somewhat.

Adequacy of arch designs. The arches designed on the above assumptions have stood without defect or showing any sign of weakness. They are thin and were made so deliberately and after much thought and study. This was recognized in the estimates submitted in the report of Hering, Stearns and Fuertes, and was discussed extensively during the preparation of that report, the conclusion being reached that every justifiable economy should be practised in the building of these works. The difficulties of putting an aqueduct through 100 miles of wild country, largely swampy, with uncertain and unknown conditions as to foundations were fully realized, and, on account of the great cost of the works for a young community grown and growing so rapidly, needing water to maintain its very existence, to have stood upon the principle of "no risk" in the designs would have rendered the project totally impossible, on account of the great cost involved.

Therefore, knowing that settlement might be expected, but that, the aqueduct being one through which the water would flow as in an open ditch without exerting pressure, there would be no difficulty and but little expense involved in making repairs, for practically the whole length the bottom of the cut-and-cover aqueduct has been placed at such an elevation that the surface of the flowing water in the aqueduct may be kept lower than the natural ground surface until the consumption of water by the District shall have increased to or exceed about 50,000,000 gallons per day. An open ditch would have served the purpose of carrying the water from river crossing to river crossing all the way from Indian Bay to Mile 23. In fact, a scheme like this was suggested in 1883 by Dr. Agnew, who proposed conveying the water to Birds Hill and distributing it from there. The cut and cover section of the aqueduct, therefore, is to be considered only in the light of a lining for a ditch, given an arched section in order to permit covering the ditch for the protection of the water from vegetation and dirt, and from cold, and to permit making the ditch smaller by reason of the smooth walls and bottom provided by the concrete surfaces, on which the water would run faster than it would in a natural ditch through the open country.

Depressed sections. Only at the river and valley crossings is this aqueduct in any respect like a pipe line, and at all such places it has been given a circular form and contains enough steel bands to take all the bursting strains due to the water pressure; and all the distortion strains due to the earth backfill, without straining the steel to more than 10,000 pounds per square inch in tension or the concrete to more than 500 pounds per square inch in compression.

These low limits were fixed after much study of the extension of the steel under tension and the amount of extension the concrete, as made, would stand without showing hair-cracks. While it was not a matter of serious consequence whether cracks appear in the cut-and-cover sections of this aqueduct, as these could be easily repaired, it would have been fatal to allow cracking of the concrete containing steel reinforcement, as the steel would then be exposed to the action of the water and would in time be attacked. As rusted steel occupies more space than clean dry steel, the exposure of the steel would result in the scaling of the concrete away from the steel and the ultimate rupture and destruction of that portion of the aqueduct. The stresses in both steel and concrete were limited, therefore, to amounts which experience gained by experimentally bursting reinforced concrete pipe lines by internal pressure showed to be well on the safe side of the danger limits, and, so far as known, to date, these assumptions have, in practice, proved sound.

DESIGNS OF INVERTS

Foundation conditions. In the design of the inverts for the different sections of the aqueduct it was, of course, recognized that the soil to be expected in the trench bottoms would vary from a semi-fluid mud to solid rock. A large percentage of the total was of boulder clay, hard-pan gravel and rock, or of soft soil on top of solid materials, as above described. The first 20 miles, however, was through a prairie country the soil of which was clay deposited from sea water ages ago, and practically devoid of silt or granular materials. This clay, however, is underlaid with a layer of gravel and boulders resting on the limestone rock some 40 feet or more below the surface. This soil is very peculiar and uncertain in its action under loads, and many serious structural accidents have occurred in Winnipeg and vicinity as a result. Its compressibility is variable, depending on its depth below the surface and on its water content.

It appears that this variability is due, in part, to the action of frost when leaving the ground, and to drainage and evaporation of its water content during the dry summer weather. Frost appears to pulverize and loosen the soil for a certain variable depth; drying cracks it open, sometimes to a depth of a few inches, in little blocks, hard in themselves, but resting on a softer base. Further drying, particularly if covered with humus or soil, sometimes opens up long continuous cracks often up to 3 inches in width at the surface, and extending down several feet; when in this condition a rain will fill these cracks, cause the ground to swell, imprison the water in its mass and force it into the lower strata, maintaining these, more or less continuously, in a condition of semi-fluidity. Often there may be a mat, or raft, of dry soil on the surface, many feet in thickness, resting on lower strata in a state of saturation.

Soil tests taken from a dressed surface, which has had a day or two to dry, may sometimes give bearing values high enough to satisfy any reasonable foundation requirement. The same soil, under other conditions as to moisture will flow, under pressure, like putty. But its worst feature is the condition of continuous, progressive settlement under a constant load, and greater relative settlements in small than in greater depths below the natural surface, under the same load.

Compressibility of clay soil at Mile 13. This is shown in tests made at Mile 13 (Deacon) in April, 1916, and abstracted in tables 5 and 6.

From the tables it is seen that doubling the load per square foot at a depth of 4 feet caused 3¼ times the settlement; doubling the load at 5 feet caused 2¾ times the settlement; and doubling it at 7 feet depth caused a settlement of 1¾ times that under the original load; also, that in all the holes and at all loads the settlement at the end of two weeks was about 1¼ times the amount at the end of the first day.

Repetitions of tests at other times showed this same general condition but never just the same total amounts. In all the above tests the ground was free from frost at the depths tested, but was moist and in its natural condition for that season of the year. When dry, on the surface, this soil would be as hard as a good road surface, and could be marked with the heel only with difficulty.

Variability in bearing powers. This variability in compressibility introduced conditions making it impossible to build on it any continuous structure that would be free from cracks when finished.

The variable depth of excavation alone would produce this, as, the weight of the structure being the same, the varying compressibility of the soil at the different depths disclosed in building up an aqueduct on a uniform grade would alone cause a wave to travel lengthwise of the aqueduct, as the backfilling progressed, resulting in transverse cracks and being a factor in the production of the longitudinal cracks. No cracks appeared in the aqueduct, either invert or arch, except under the weight of the backfill or the action of frost on unprotected work. Generally none showed with backfills of 2 feet or less.

TABLE 5

Settlement of clay subsoil under a load of 3000 pounds per square foot

DEPTH OF TEST AREA BELOW NATURAL SURFACE	TOTAL SETTLEMENT IN FEET AT END OF DAY NUMBER												
	1	2	3	4	5	6	7	8	9	10	11	12	13
feet	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*
4	0.023	0.024	0.025	0.028	0.028	0.028	0.031	0.032	0.032	0.032	0.032	0.032	0.032
5	0.027	0.021	0.029	0.029	0.030	0.030	0.032	0.024	0.035	0.035	0.035	0.035	0.035
7	0.017	0.018	0.018	0.019	0.020	0.020	0.020	0.021	0.021	0.021	0.021	0.021	0.021

TABLE 6

Settlement of clay subsoil under a load of 6000 pounds per square inch

DEPTH OF TEST AREA BELOW NATURAL SURFACE	TOTAL SETTLEMENT IN FEET AT END OF DAY NUMBER												
	1	2	3	4	5	6	7	8	9	10	11	12	13
feet	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*	*feet*
4	0.077	0.090	0.098	0.101	0.103	0.103	0.103	0.103	0.103	0.103	0.103	0.103	0.103
5	0.076	0.079	0.081	0.081	0.082	0.083	0.084	0.086	0.086	0.086	0.086	0.086	0.088
7	0.030	0.031	0.032	0.033	0.034	0.035	0.035	0.035	0.036	0.037	0.037	0.037	0.037

These conditions were foreseen and fully weighed before proceeding with the design and construction of the inverts. Cracks were expected in this part of the work.

Further east, soft bottom was also known to exist in a number of locations of short lengths. These were, during construction of the work, either entirely dug out and refilled with water-settled sand and gravel, fills, or else rolled embankments; or pile foundations were used. A very few unimportant cracks occurred in the inverts in a few localities so treated.

Special inverts for varying conditions. In some places, particularly where roads and railroads crossed the aqueduct, the section was thickened and reinforced with steel bars to distribute the weight more uniformly over the entire width of the base, to prevent excessive local settlement. Also, where ground water levels were so high as to tend to float the aqueduct, when empty, and where the backfilling materials were very light in weight, the aqueduct bottom was thickened sufficiently to add weight enough to resist flotation of the tube, and the bottom and arch reinforced, as required, against water pressures from without and within, as well as for backfill loads. All these contingencies were foreseen and provided for in the preliminary estimates of cost. No special allowance, however, was made in these estimates for pile foundations, or other means to secure a rigid foundation for the work, through the prairie section above referred to, as it was the judgment of the engineers making the original report that this would not be necessary.

Settlement cracks. As a matter of fact, troubles from settlement occurred only in about 1.6 miles of the 20 miles of contract 30 of this treacherous country; 0.45 mile of the 17¾ miles of contract 31, 0.2 mile of the 18.2 miles of contract 32; about 0.2 mile of the 16.1 miles of contract 33, and about 0.02 mile of the 13 miles of contract 34 by September, 1916, practically all the cracks having been in the 1915 works. This date is given because subsequently a much more expensive form of construction was adopted to prevent such cracks, the changes consisting of using 8-inch extensions or footings on the edges of the invert, to reduce the maximum pressures to lower amounts on poor foundations, and using a heavy, thicker invert, reinforced with steel, where the ground was less hard than the best.

Repairing cracks. None of the work above referred to as cracked was taken out or replaced. Most of it was repaired by cutting out the cracks to a width of about half an inch for a depth of 2 inches and packing the cracks with hammer-calked neat Portland cement, put in very dry and tamped very solidly, first washing all loose chips, sand and dust out of the prepared cracks with water under pressure. This method of repair made a very strong tight joint; in fact, the edges of the crack were knit together as strongly as though they had never been broken. Several sections of invert, including portions of these joints, have been cut out and tested. The cement filling, even after having been in place only two or three days, adheres so strongly to the concrete that, when broken, the break was generally in the old concrete, sometimes in the filling, but never at the junction of the two.

Water tightness of repaired work. Also, to test the tightness of the aqueduct after repair, a number of sections were selected and tested. One of these, 270 feet long, was purposely picked out to include the most seriously damaged portion. In this section the invert had several longitudinal cracks, some open ½ inch at the surface, others of smaller widths, and the arch was cracked along the haunch on one side as well as along the top. Bulkheads were built at each end of the section, which was then filled with water to the level corresponding to a rate of discharge of 85,000,000 gallons per day. Daily records were kept of the drop in water-level for some months and the rate of leakage found to be less than 5,000 gallons per mile of aqueduct per day, which for this size of aqueduct, equivalent to a circle 8.1 feet diameter, would hardly be called excessive for a cast iron pipe line with calked joints; the average of the other tests gave a leakage of about one-third that amount.

Cost of repairs. The actual cost of the repairs of the 5300 feet repaired, including labor, materials and incidentals, was less than half a dollar per foot of aqueduct repaired; in fact, about $2,500 in all.

With the greater experience gained both by the engineers and inspectors and by the contractors' men, in securing more thoroughly compacted foundations, particularly along the sides of the inverts, practically all invert cracks could have been eliminated in future work without the use of heavy reinforced inverts.

Types of inverts used. The standard invert used throughout during the 1915 construction program, and until April, 1916, was a concrete slab as wide as the extreme spread of the feet of the arch, from outside to outside, with a curved upper surface, struck to a radius of 14 feet for the aqueduct 9 feet high, the largest size used, and 8 feet

1¼ inches for the aqueduct 5 feet 4¾ inches high, the smallest size used. The slab was 6 inches thick in the center and for a certain distance each side, then built on a uniform flat slope to the full width required, and was laid in 15-foot lengths, with crimped copper expansion joints between each slab, in advance of and to form a footing for the arches. During the first year's work it was next to impossible to get the contractors' men to consolidate the foundation properly along the edges of the trench, particularly where the soil was of clay, the temptation being so strong to trim the surface to the exact grade required with sharp shovels, which cut it like cheese and left a fine surface for the concrete. This operation, however, left the top surface uncompacted and capable of considerable compression under moderate loads, and a large percentage of the fine cracks in the invert were due to this fact. Finally it was required that all compressible clay sub-grades, particularly in shallow cuts and where the trench was moist or wet, should be covered with a thin layer of gravel, to be heavily hand-rammed for a width of about 3 or 4 feet along the outside edges. When it is realized that a deflection of only about one-thirtieth of an inch would cause cracks in the inverts, the reason for a hair line crack along the center of the invert when the back fill was put over the arch is easily understood where the sub-grade was simply trimmed with a shovel. The uncompacted soil would at times exhibit that much compression under loads of only 700 pounds per square foot, or about 5 pounds per square inch. Rolling these sub-grades even under planks on a gravel bed was impracticable; it would jelly below the surface and work into waves ahead of the roller; hand compacting by heavy rammers on a thin bed of gravel seemed to be the best means of getting hard enough bottom; and when this was done properly and conscientiously, as it was after the first season's work had shown the contractors and inspectors the results of laxity in this regard, the hair line cracks from this cause were practically eliminated.

The large cracks were due to a different cause, the compressibility of the soil by greater amounts. The only way to avoid these cracks was to reduce the unit pressure under the arch footings by spreading the invert to a greater width, or to thicken and reinforce the bottom of the aqueduct so as to distribute the pressure over the whole bottom width of the invert, as uniformly as practicable.

Testing different types of invert to destruction. In order to have something more than theory as a guide as to the actual distribution of pressure over the bottom of the aqueduct, to aid in proportioning the inverts for this condition of soil, a series of full-size inverts for the 8-foot 9-inch by 7-foot 4⅝-inch aqueduct, made 2 feet in width and laid parallel to each other on the natural soil trimmed to shape, was built at Deacon. There were in all 16 inverts, as follows:

A1, A2 and A3. Standard inverts 6 inches thick in the middle, as used in all aqueducts prior to April, 1916.

B. Standard invert with 3 wood strips ⅝ by 1½ by 24 inches, built across the invert in the center and near the ends, where cracking had occurred in the construction work.

C1 and C2. Standard inverts 6 inches thick at center with 12-inch extension on each end, making the out-to-out width 14 feet instead of 12 feet as in the original inverts for this size of aqueduct.

D1 and D2. Standard inverts 6 inches thick reinforced with ½-inch bars 8 inches center to center.

E1 and E2. Standard inverts 6 inches thick reinforced with ⅝-inch square bars 8 inches center to center.

F1 and F2. Standard inverts 7½ inches thick reinforced with ⅝-inch square bars 8 inches center to center.

G1 and G2. Invert of standard width, but 10 inches thick, reinforced with ⅝-inch square bars 8 inches center to center.

H. Invert of standard width, but 11¼ inches thick, reinforced with ⅝-inch square bars 4 inches center to center.

K. Standard invert 6 inches thick reinforced with ⅝-inch square bars 4 inches center to center.

These inverts were built in the regular way and were provided with concrete pedestals about 2 feet high on each side, of the width of the sides of the arches. The loading was done by laying 60-pound steel rails across the top of the two pedestals and observing the settlement of the invert at each side, at the middle and at the quarter points, as the load was increased. The deflections or settlement, as reported, were measured with accuracy below a fine bronze wire stretched tightly across, from posts driven deeply into the ground each side of each test invert.

The load and deflection at each point were recorded as each crack appeared, and from these records a number of important deductions were evident.

Practical deductions from tests. For instance, in inverts C1 and C2 cracks did not appear until the load carried was as great as that required to produce the first cracks in inverts F1 and F2, which were

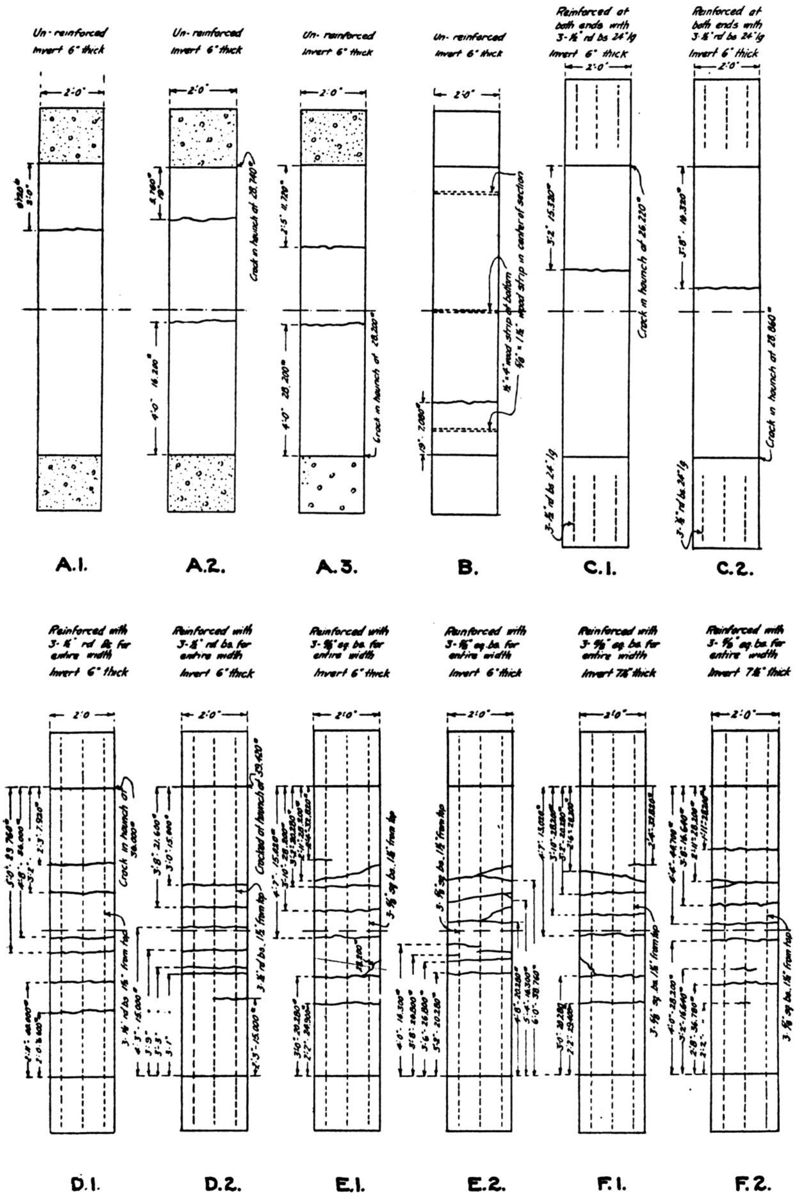

Un-reinforced
Invert 6" thick
Reinforced at both ends with 3-½" rd bs 24" lg
Invert 6" thick
2'-0"
Crack in haunch at 28,740#
Crack in haunch at 28,200#
¾" × 4" wood strip at bottom
⅝" × 1½" wood strip in center of section
Crack in haunch at 26,220#
Crack in haunch at 28,860#
3-½" rd bs 24" lg
A.1.
A.2.
A.3.
B.
C.1.
C.2.
Reinforced with 3-½" rd bs for entire width
Invert 6" thick
Reinforced with 3-⅝" sq bs for entire width
Invert 7½" thick
Crack in haunch at 36,000#
Cracked at haunch at 33,420#
3-½" rd bs 1½" from top
3-⅝" sq bs 1½" from top
D.1.
D.2.
E.1.
E.2.
F.1.
F.2.

FIG. 7A. POSITION OF CRACKS IN TEST INVERTS AND LOADING CAUSING EACH CRACK

$1\frac{1}{2}$ inches thicker than C1 and C2; and in addition were reinforced with $\frac{5}{8}$-inch square bars 8 inches center to center. In fact, those two thin, unreinforced inverts carried a greater load, without a crack, than any other except those which were heavily reinforced and nearly twice as thick. There was, however, this important difference, that by the time the test load had reached an amount equivalent to the actual load due to the aqueduct and its backfill and water load, a little over 5000 pounds per side per foot, the inverts C1 and C2 had shown but one crack each in the center, which load had to be increased by 50 per cent to cause the second crack to appear at the edge of the invert in each case. Reinforced inverts D1, D2, E1, E2, F1 and F2, showed from 2 to 6 cracks under the same loading as C1 and C2; and even G1, G2 and K showed 2 to 5 cracks across the surface before the load reached the amount normally to be carried by the arch.

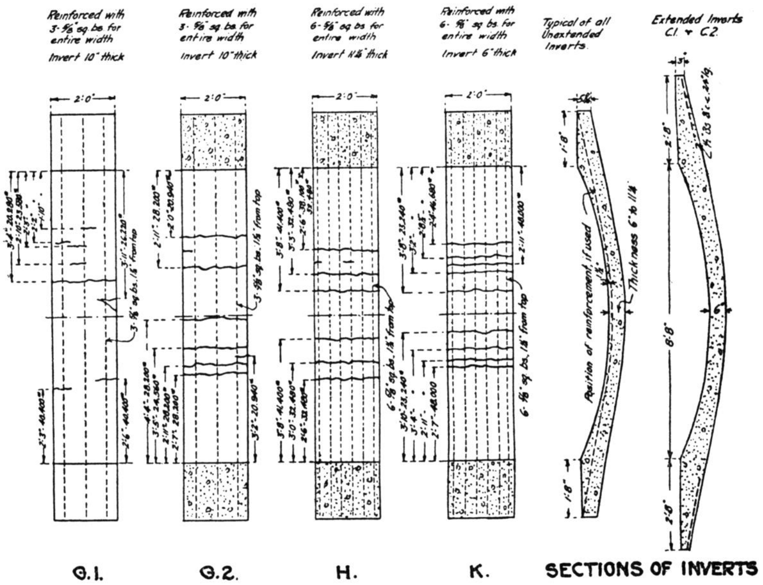

Reinforced with 3-⅝" sq. bs. for entire width
Invert 10" thick
Reinforced with 3-⅝" sq bs for entire width
Invert 10" thick
Reinforced with 6-⅝" sq bs for entire width
Invert 11¼" thick
Reinforced with 6-⅝" sq bs for entire width
Invert 6" thick
Typical of all Unextended Inverts
Extended Inverts C1 + C2
2'0"
2'0"
2'0"
2'0"
1'8"
Position of reinforcement, if used
Thickness 6" to 11¼
2'8"
6'8"
G.1.
G.2.
H.
K.
SECTIONS OF INVERTS

FIG. 7B. POSITION OF CRACKS IN TEST INVERTS AND LOADING CAUSING EACH CRACK

In other words, the only reinforced invert that demonstrated its superiority to plain thin inverts C1 and C2 with an extended base was invert H, which was twice as thick ($11\frac{1}{4}$ inches) and also reinforced with ⅝-inch square bars 4 inches center to center. Furthermore, the cracks in inverts C1 and C2 could be repaired and the results be made as good as the originals, while the reinforced inverts could not be repaired except at much greater cost, owing to the multiplicity of cracks, and at the risk of further disintegration if acid soil water should penetrate to the steel from below.

The appearance of these 16 inverts, with the location of the cracks and the loads under which they occurred, is shown in figures 7A and 7B.

To prevent further cracks in the invert, a change was made, early in 1916, by adopting for all solid ground, regardless of whether it were compressible or not, a standard invert 6 inches thick at the center with an 8-inch extension on each side beyond the standard invert, and for all questionable foundations, inverts varying from 8¼ to 14½ inches in thickness, reinforced with ⅝-inch bars from 4 to 5¾ inches center to center, according to the size of the section. After the tests above referred to were completed and others, also, along different lines, a more conservative program was adopted, as follows:

Types adopted for construction after 1916. Three types of invert were designed, to cover the range of variability in foundations. Type A was for compressible soils; type E for poorer than best but better than worst foundations, and type O for solid foundations.

The invert adopted for solid foundations was the original design used throughout 1916.

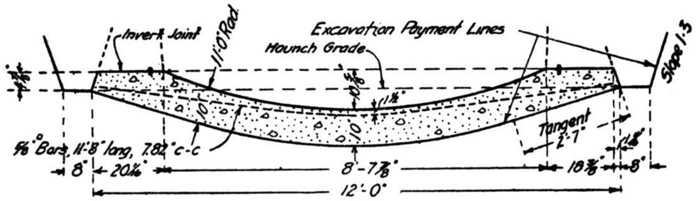
For the poorer than best foundations the standard invert of 1915, extended 8 inches on each side, was used.

For the compressible foundations the inverts were the standard inverts of 1915 thickened from 6 inches to from 7½ to 12¼ inches, and reinforced with ⅝-inch bars from 7½ to 9 inches center to center, according to the size of the section. This policy resulted in a considerable saving over the extravagant policy of 1916.

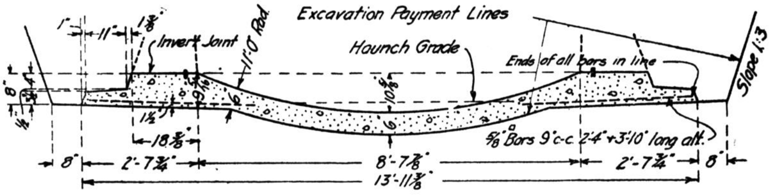
The general designs for these three types, for the B section of the aqueduct (8 feet 9 inches by 7 feet 4⅝ inches) are shown in figure 8.

While the use of these expensive inverts stopped the occurrence of longitudinal cracks in the inverts, which seemed to be only ones that excited comment and criticism from without, they did not, in the least, stop the formation of transverse cracks in the arches; in fact, these transverse cracks could not be stopped by any means except to put the aqueduct on a pile foundation throughout the country with compressible soil.

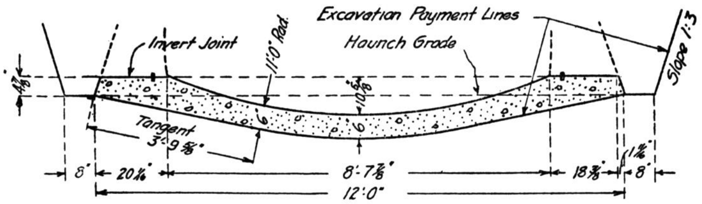
The aggregate length of transverse cracks in the aqueduct arches, some caused by temperature, because of delaying the backfilling till too late in the season, but most of them by the plan used by the contractors for backfilling, probably exceeds the lengths of the longitudinal cracks by a very large percentage.



SECTION LETTER BA



SECTION LETTER BE



SECTION LETTER BO

FIG. 8. INVERTS FOR AQUEDUCT 8 FEET 9 INCHES WIDE AND 7 FEET 4⅝ INCHES HIGH

The author attached little importance, so far as the integrity of the work was concerned, to either the longitudinal or transverse cracks, as the cost of preventing these would far exceed the cost of repairs, and the repaired work, being under no strain, would remain in equilibrium in the future.

Precautions to prevent flotation of aqueduct. As to invert designs but one more condition required consideration and that was for use where the available backfill material was of light weight, the aqueduct submerged and the foundation soil porous, or more than a certain depth below grade, varying from 25 inches for the 8-foot 9-inch by 7-foot $4\frac{5}{8}$-inch section to 31 inches for the 10-foot 9-inch by 9-foot section. The problem here was to secure stability and permanence for the aqueduct, full and empty, when the ground water level was above the arch of the aqueduct and the backfill materials so soft, light and lacking in cohesive properties as to be of practically no value except for frost-proofing. Many studies were made of plans to secure this result, such as gravel backfill, hauled from the District's pits by train, backfills placed by hand, and thin reinforced inverts covered with two or three feet of packed graded gravel, and having a concrete paving over the gravel to form a smooth durable invert. The most economical and best plan was the one shown in figure 9.

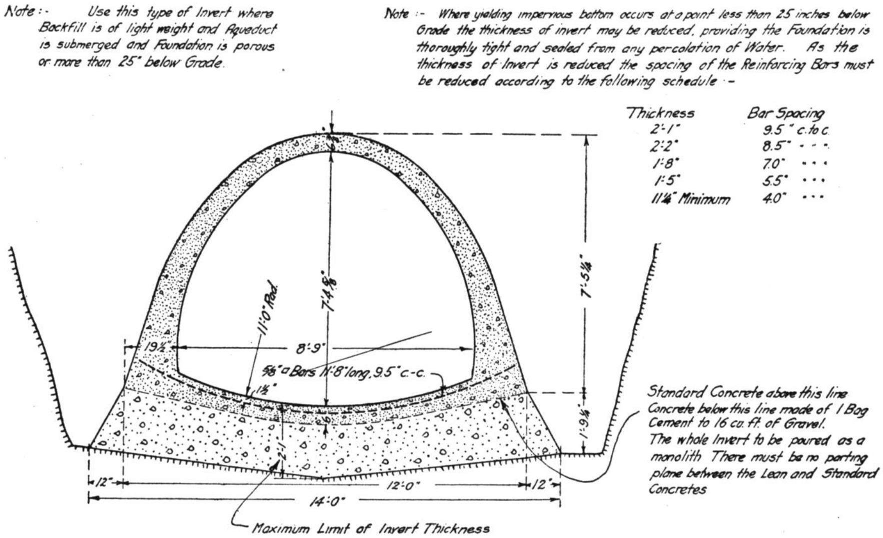

Note :- Use this type of Invert where Backfill is of light weight and Aqueduct is submerged and Foundation is porous or more than 25" below Grade.
Note :- Where yielding impervious bottom occurs at a point less than 25 inches below Grade the thickness of invert may be reduced, providing the Foundation is thoroughly tight and sealed from any percolation of Water. As the thickness of Invert is reduced the spacing of the Reinforcing Bars must be reduced according to the following schedule :-
Thickness Bar Spacing
2'-1" 9.5" c. to c.
2'-2" 8.5" " " "
1'-8" 7.0" " " "
1'-5" 5.5" " " "
11¼" Minimum 4.0" " " "
7'-4⅝"
7'-5¼"
11'-0" Rad.
19½"
8'-9"
⅝" ⌀ Bars 11'-8" long, 9.5" c.-c.
1½"
6"
2'-1"
1'-9¼"
12"
12'-0"
12"
14'-0"
Maximum Limit of Invert Thickness
Standard Concrete above this line Concrete below this line made of 1 Bag Cement to 16 cu. ft. of Gravel. The whole Invert to be poured as a monolith. There must be no parting plane between the Lean and Standard Concretes

FIG. 9. SPECIAL INVERT FOR SECTIONS TO RESIST FLOTATION

Acid soil. As has been already mentioned the nature of the processes resulting in the formation of the prairie sections, from Winnipeg eastward to the gravel deposits from the glacial drift about Mile

25, has left the soil impregnated with the salts formed from the evaporation of the sea which once covered this area. The country is so flat that these have never been washed out of the soil, except locally, and there has been, rather, a tendency towards concentration and combination with the soil elements to form compounds which, in the presence of moisture, will combine with the lime in the concrete to form sulphur and chlorine compounds of lime. The sulphur compounds, particularly, are objectionable, because they soften the concrete, if porous at all, and gradually eat into it.

The presence of these salts was found by analyses of many samples in soils at various parts of the line, but not of sufficient concentration to indicate a troublesome action anywhere except certain places west of about mile 30.

Protection of aqueduct. The remedy adopted for this trouble was thorough drainage of the soil to prevent the concentration of the salts locally. The whole aqueduct from mile 17 eastward to Shoal Lake, and the pipe lines from Mile 13 westward to Winnipeg, are protected in this regard. The 4 miles, from mile 13 to mile 17, which is an 8-foot diameter section, with very thick walls, built in place, depressed so low below the prairie that drainage could not be had for the bottom of the trench to any nearby watercourse, and which was the first pressure pipe to be built on the aqueduct, was not underdrained. By an accident of chance the excavation was made when the soil was relatively dry so that the invert could be trimmed exactly to fit the concrete, in the dry, and hence the underdrains leading to sumps that would have been required had the trench been wet, were not put in, and no way exists to drain the soil here without an expensive construction program.

Also, it was assumed that the use of dense hard concrete, carefully placed and with a smooth surface, would resist the disintegrating action of the soil, if the surface waters were properly taken care of, so that there would be no further concentration of the salt locally; and that the actions, if any, would be limited and gradually disappear before damage worthy of attention could be done, by the leaching out of the soil. Along parts of the line, however, in these 4 miles, examinations of the pipe that had been in the ground two years showed, in places, a limited softening of the concrete on the surface, not continuous, as to areas attacked.

REINFORCED CONCRETE PRESSURE PIPES

The premoulded reinforced concrete pipes from Deacon to the reservoir in Winnipeg were built by the Canadian Lock Joint Pipe Company, under general specifications prepared by the District.

The 5½-foot diameter pipe, from Deacon to Red River, 49,900 feet long, was built for and tested in one length to the full head caused by the water in Deacon Reservoir (proposed) with the water shut-off at the Red River, plus a few feet for mild surging. The limiting leakage allowed by the specifications under the above head was at the rate of 115,000 imperial gallons per 24 hours for the 49,900 feet. The actual test showed a safe margin below the requirements.

The 48-inch diameter pipe, from Red River to McPhillips Street Reservoir, 11,400 feet, was built for and tested as a unit, after laying, to a pressure of 40 pounds per square inch. The limiting leakage allowed by the specifications was 9000 gallons per mile per day, and there was, as with the larger pipe, a safe margin below the specified limit.

The pipe was built at Transcona, shipped to the trench and laid by special machinery and methods. The joints were all provided with crimped copper strips, as water cut-offs, every 8 feet for the 5½-foot pipe and every 10 feet for the 48-inch. The methods of manufacture, transporation and laying of this pipe were so well worked out and under control that of the total length of 61,300 feet manufactured (barring the three lengths of 5½-foot-pipe made for experimental purposes by the District, one broken in loading and one purposely broken by pressure in testing), there were only 2 culls on the 5½-foot pipe and 2 on the 48-inch pipe.

DESIGNS FOR SPECIAL STRUCTURES

Overflows and blow-offs. Mention has hereinbefore been made of the use of overflows at all the important river crossings to prevent accidents to the arch section by careless operation of the gates at the intake. In figure 10 are given a plan and section of the overflow at Boggy River, which may be taken as typical.

The main features of these structures are the provision of ample overflow facilities, a blow-out valve, stoplogs to block or regulate the depth of flow in the aqueduct, a boat entrance for taking out boats used for inspection and floating down stream from the next

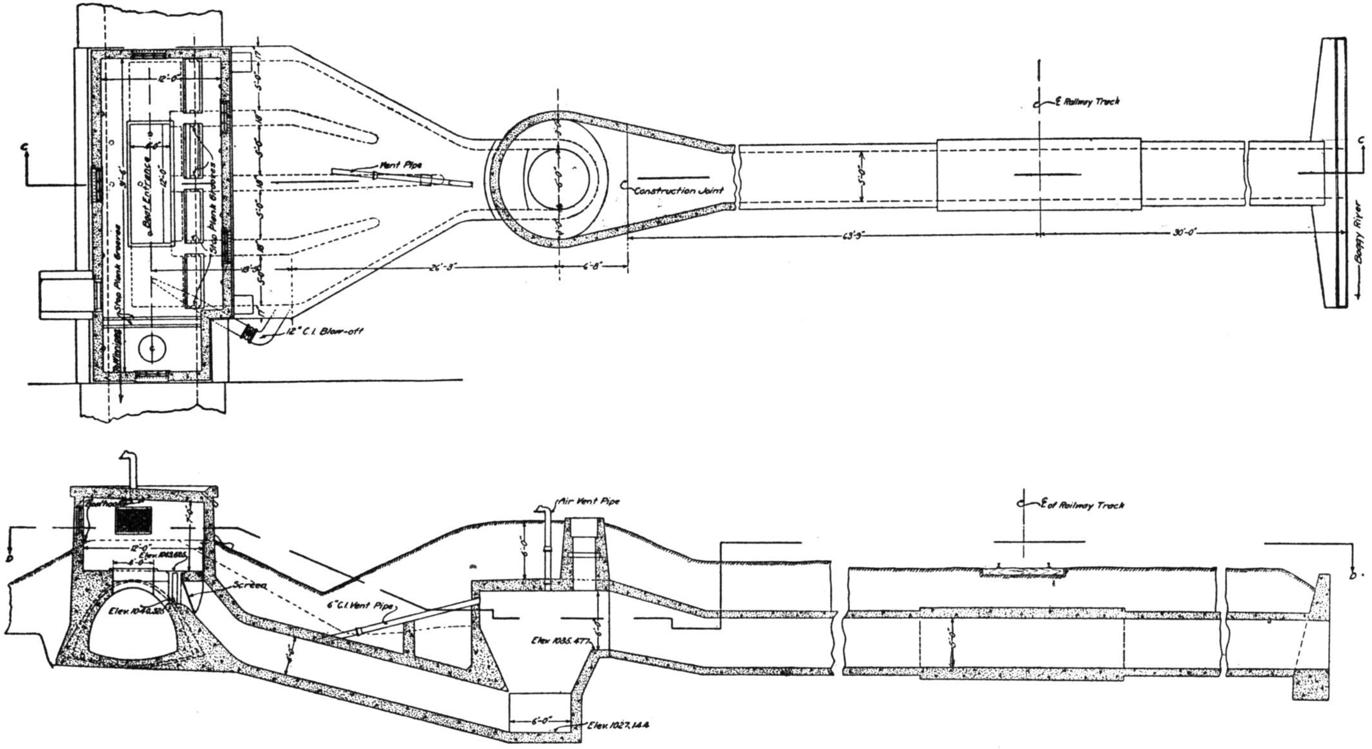


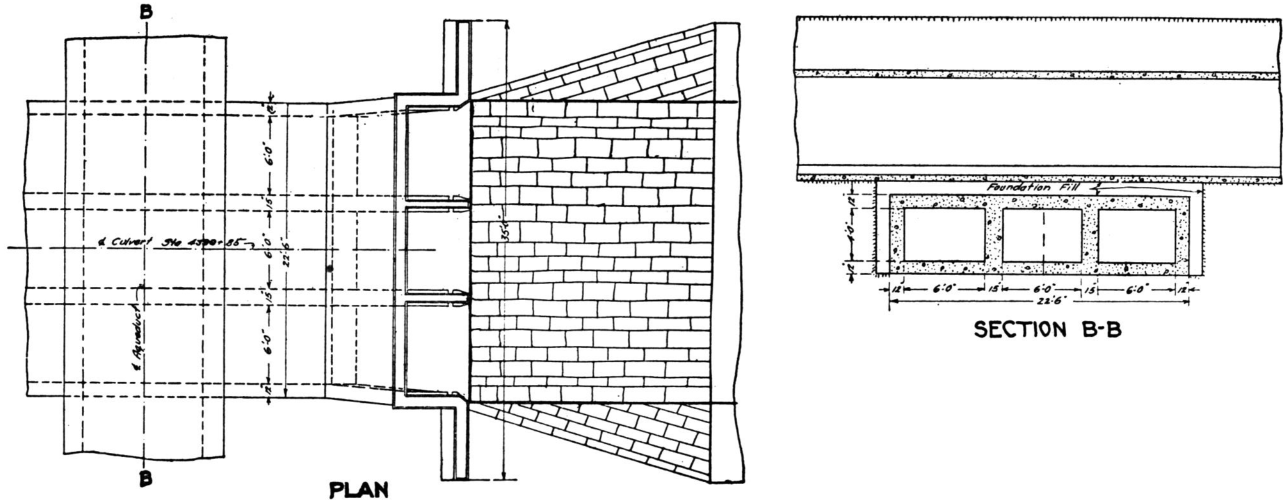
FIG. 10. OVERFLOW AND BLOW-OFF AT BOGGY RIVER

entrance upstream; an overflow channel to the river, arranged in such manner as to exclude the cold outside air from the interior of the aqueduct, and remain open at all times for the discharge of the overflowing water; provisions for balancing the air pressures in the aqueduct, due to changing depths of flow, etc., and a house covering the aqueduct where the overflow is located.

Culverts under the aqueduct. To take care of surface drainage crossing the line of the aqueduct culverts of different sizes were required at various points. Except at the major river crossings, surface drainage is taken under the aqueduct, owing to the elevation of the grade line relative to the ground surface, in an inverted siphon. The details of these structures, shown in general on figures 11 and 12, are similar in all cases, varying only in size or details to adapt them to various locations.

The form adopted allows of cleaning these out, if necessary, from time to time, and provides frost proofing for all parts except the open mouths, which are so designed as to be easily freed from ice forming there in very cold weather The outlet and inlet ends are provided with stop-log grooves to permit blocking out the water for cleaning or removing ice, and have heavy chains hung across the entrance to keep out wandering animals, such as bear, moose, and caribou, as well as cattle in the few localities where there are any running loose in the woods. The top of the conduit, where it passes under the aqueduct, is placed lower than the bottom of the outlet ditch to seal out the cold air which might, if entering, cause trouble with the aqueduct invert.

Miscellaneous small works. Mention, merely, is necessary of the provisions for air valves on the closed pipe lines, and air vents for the valve chambers and other structures where great cold might work damage to the valve, etc., housed therein. In some places, notably on the 5½-foot concrete pipe line, special vents were devised, consisting of a vent pipe with the lower end immersed slightly in a shallow pan of non-volatile oil, an expedient which would permit maintaining equilibrium of pressure without free circulation of the cold air.



PLAN

SECTION B-B

Fig. 11. Details of Culvert at Station 4399 + 85

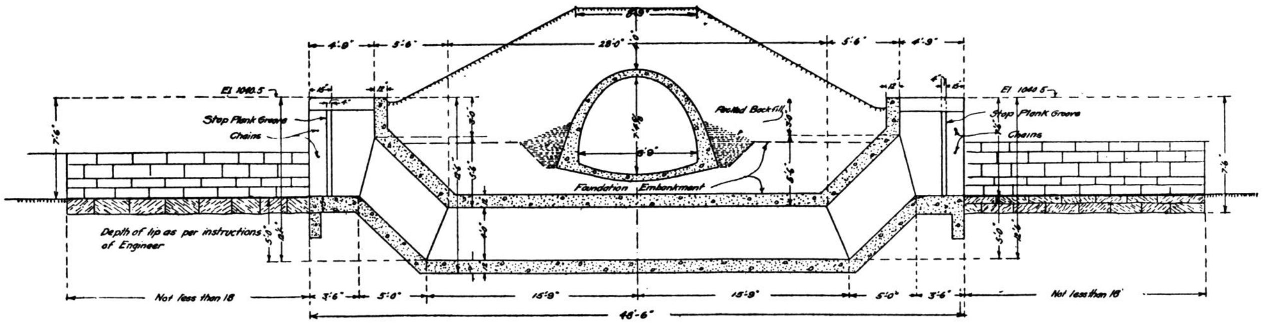


Fig. 12. Section Through Culvert at Station 2399 + 85

CONCLUSION

The word of caution incorporated in the report of Hering, Stearns and Fuertes that "The cost of the work will also depend, to a great extent, on the skill exercised in the final location of the line and the establishment of the grades in a proper manner. A margin of half a million dollars, or more, may easily be used up by failure to appreciate the elements in which economies may be practised," can be appreciated by mention of the fact that had the arches and inverts been made but 1 inch thicker than they were there would have been required some 35,000 yards of extra concrete at a total extra cost of approximately $500,000.

There was, therefore, no margin for "slap-dash" methods of designing; all things had to be considered with great care to prevent extravagance. The bases of design had to be sound that the actual works could be pared down to the narrowest margin of excess possible, or the money would vanish in half-million dollar blocks for every extra dollar per foot the work cost.

It is worthy of note, now that the aqueduct has been completed and is in its second season of operation, that all of the portions of the 1915 work that suffered settlement cracks, when the backfill was placed on them, are still in place and in use, having been repaired, as above described; and that the aqueduct, owing to the smoothness of its interior surfaces and the excellence of the finish, both as to lines, grades and uniformity throughout, will, despite predictions to the contrary and despite the cracks, not one half of which were repaired at all, deliver in its present condition more water than the conservative capacity assigned to the scheme in the Hering, Stearns, Fuertes report. Further, the total cost of the construction features of the whole proposition has not exceeded, including the construction railway and all extra work and repairs, the original estimates of Hering, Stearns and Fuertes, the allowances that were made in those estimates to cover extra foundation costs, ditching, swamp drainage, the construction railway and equipment, the probable extra quantities of concrete, excavation, steel for reinforcement, difficulties at stream crossings, and other elements difficult to foresee, having proved sufficient to cover just such contingencies as were encountered. Further, also, these exceptionally fine results were achieved in spite of war conditions, the plans for the aqueduct having been completed early in 1914, and the construction of the railway well advanced before the war broke out; and the water of Shoal Lake was delivered into the Winnipeg Reservoir in the spring of 1919. But for the fear, locally, of trouble with steam boilers and heating plants as a result of changing suddenly from a very hard to a soft water in the midst of cold weather, the new water could have been made available in the winter of 1918, so far as the aqueduct and pipe lines were concerned.

While the rated capacity of the aqueduct is generally understood to be about 85,000,000 imperial gallons per day, its actual capacity, clean, at the depth of flow giving the greatest discharge, is not much under 100,000,000 gallons per day; even so, however, without the proposed Deacon reservoir in use, the effective capacity of the works, from Shoal Lake to Deacon, will take care, only, of a population using an average of about 60,000,000 gallons per day, and in delivering this quantity there will be risk to the aqueduct unless operated under the care and direction of an experienced man, thoroughly posted, both technically and practically, who knows how to control the flow to prevent surges when the velocity changes from low to high rates, and vice-versa, under the exigencies of service conditions. With the Deacon Reservoir in service the danger to the aqueduct will disappear and its capacity will automatically increase 25 per cent, figured on the average daily demand for water. It is evident, therefore, that as soon as boosting becomes necessary at Red River (which will be done by pumps discharging the water at a rate of 50,000,000 gallons per day), the Deacon Reservoir should be in service; if it is not, the regulation of the flow in the gravity section of the aqueduct, east of Deacon, will present problems of some difficulty, as the frequent slowing down of the flow to an average rate of 1.5 feet and sudden speeding it up again to 4 feet per second, in response to booster demands, will require an expensive organization, or the continuous wasting of a much larger quantity of water at the overflow at the beginning of the 8-foot circular depressed section, at station 900+30, than can be accommodated there without subjecting private property in the vicinity to occasional floodings.

In closing this very superficially descriptive paper dealing with some of the novelties of design in connection with the works of the New Greater Winnipeg Water Supply, the author would be presuming far beyond his province to attempt to apportion the credit for the success achieved. A deep appreciation of his own obligations, however, no less than a sentiment of high personal regard, pels him to offer here a few words of tribute to all his superiors and associates, and particularly to Thomas R. Deacon, C.E., former mayor of Winnipeg and first chairman of the Administration Board, who was always a most consistent champion of Shoal Lake as a source for Winnipeg's future water supply, and whose keen foresight, inspiring leadership, great energy and tireless exertions turned what seemed a highly desirable possibility into a practical reality; and to the late S. H. Reynolds, C.E., first chairman of the Board of Commissioners, and James H. Ashdown, Esq., Commissioner, whose unselfish devotion to the interests of the undertaking laid the foundations never afterward to be disturbed for the smooth, orderly and properly controlled development of the project.